Effect of Temperature on Swelling Pressure and Compressibility Characteristics of Soil

Ahsan Rabbani

Department of Civil Engineering

National Institute of Technology Rourkela

Effect of Temperature on Swelling Pressure and Compressibility Characteristics of Soil

A thesis submitted in partial fulfillment of the requirements for the degree of

Master of Technology

in

Geotechnical Engineering

by

Ahsan Rabbani

(Roll No: 214CE1042)

based on research carried out under the supervision of

Prof. Ramakrishna Bag

Department of Civil Engineering

National Institute of Technology Rourkela

Department of Civil Engineering

National Institute of Technology Rourkela

May 25, 2016

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Roll Number: *214CE1042*

Name: *Ahsan Rabbani*

Title of Thesis: *Effect of Temperature on Swelling Pressure and Compressibility Characteristics of Soil*

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Prof. Ramakrishna Bag
Supervisor

Head of the Department

Department of Civil Engineering

National Institute of Technology Rourkela

Prof. Ramakrishna Bag

Assistant Professor

May 25, 2016

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I, *Ahsan Rabbani*, Roll Number *214CE1042* hereby declare that this thesis entitled *Effect of Temperature on Swelling Pressure and Compressibility Characteristics of Soil* presents my original work carried out as a M Tech student of NIT Rourkela and, to the best of my knowledge, contains no material previously published or written by another person, nor any material presented by me for the award of any degree or diploma of NIT Rourkela or any other institution. Any contribution made to this research by others, with whom I have worked at NIT Rourkela or elsewhere, is explicitly acknowledged in the thesis. Works of other authors cited in this thesis have been duly acknowledged under the section "Reference". I have also submitted my original research records to the scrutiny committee for evaluation of my thesis.

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Ahsan Rabbani

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May 25, 2016
NIT Rourkela

Ahsan Rabbani
Roll Number: 214CE1042

Abstract

Construction of building on clay soil is highly risky due to its poor strength. Clayey soil creates many problems to the Geotechnical Engineers primarily because of repeated change of moisture content. Normally, these soils increase in size and swell when they absorb water and reduce in size and shrink when they become dry. Volume change in soil leads to distortions in the form of settlement due to contraction as a result of dryness or in the form of expansion due to swelling as a result of absorption of water. There may be need for treatment of these soils to improve their engineering properties. Bentonite in compacted form is mainly used as a buffer material for nuclear waste disposal system. A better idea of the hydro-mechanical behavior of clay soil is essential to ensure safe disposal.

The present study reports the results on the effects of temperature on swelling pressure and compressibility characteristics of soil. In this project work, two different type of soils were used. One of them was a bentonite (liquid limit = 139%) procured from Bikaner, Rajasthan another one was Rourkela local soil (liquid limit = 35%). A new product modified oedometer was designed and developed in-house to carry out consolidation and swelling pressure tests at a higher temperature. Swelling pressures tests on compacted bentonite specimens of targeted dry density of 1.6 Mg/m^3 were conducted under constant volume condition for the temperature range between 25 to 90 ^0C. Compressibility tests at various temperatures for both soils were conducted using distilled water as the saturating fluid.

It observed that high temperature caused an increase in swelling pressure and compressibility index of bentonite soil. There is no effect of temperature on compressibility index of Rourkela local soil.

Keywords: Compacted bentonite; Local soil; Temperature; Swelling; Swelling pressure; Compressibility.

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Chapter1

Introduction

1.1 Background and motivation

An expansive soil can be identified by the potential of the soil to swell independently of field conditions such as water content and surcharge pressure. Because of swelling behavior properties, these types of soil produces problems like settlement and foundation crack. Many times these problems become unfavorable. The plasticity index of soil is the main indicator whether the soil is expansive in nature or not. Excess shrink or swelling behavior of these soils make the ground slope unstable and cause the unfavorable problem to the building foundation. Most designs of geological repositories constructed at a greater depth for nuclear wastes done using multi-barrier concept, which includes the natural geological barrier, engineered barriers made from compacted bentonite and the metal canisters. The natural geological barrier is nothing but host rock. Compacted bentonite-based materials are relevant materials for the purpose of the barrier and backfilling material in the waste repository, because of their high swelling, high radionuclide reduction capacity and low value of permeability (Yong et al., 1986; Villar and Lloret, 2008). Stability of nuclear waste disposal repository mainly depends on swelling pressure of compacted bentonites (Tripathy et al., 2014).

Bentonites in compacted form preferred as a barrier and backfilling material in the waste disposal repositories. The reason is to (*i*) hold the nuclear waste canisters structurally in place and (*ii*) increase the water tightness and at the same time it transfer the generated heat by the waste to the host rock (Tripathy et al., 2004). At the time of saturation and under the confined boundary conditions the bentonites in compacted form produces swelling pressures. The section of bentonite that is in contact with waste canisters is subjected to elevated temperature upon receiving heat from the radioactive waste, whereas the other section of the compacted bentonite in contact with saturated host rock receive groundwater from the saturated host rock. Once in operation, the host rock is expected to serve as the source of hydration for the bentonites which is in compacted condition and also it act as a confinement against the volume change because of hydration process. On the other hand, a bentonite buffer near the canister will initially dry and shrink due to receiving heat

induced by the canister. Because of the complexity of the repository, interests on the long term behavior of the bentonite buffers have increasingly gained recognition (Villar and Lloret, 2004).

The change in the water content of the soil due to temperature change is the crucial issues in geoenvironmental engineering (Villar and Lloret, 2004; Delage and Romero, 2008). Most problems associated with the behavior of clay liners and backfill materials are related to the change in volume because of variations in the moisture content (Delage and Romero, 2008). The volume change behavior of fine-grained soil during drying and wetting influenced by the physical and chemical forces (Bolt 1956; Mitchell, 1993; Tripathy and Schanz, 2007; Schanz and Tripathy, 2009). Not only physicochemical force, the microstructural arrangement of the clay particles and frictional forces are also important in controlling the behavior of soils (Mitchell, 1993; Delage et al., 2006). Therefore, understanding the behavior of compacted bentonites under thermal, hydraulic and mechanical loading necessary for the long-term safety of the nuclear waste disposal repository. Also at this time, it is very important to study the variation of moisture content, temperature, and degree of saturation in compacted bentonite.

1.2 Objective of the research works

Main objectives of the research work are

1. Development of modified oedometer for conducting swelling pressure and consolidation test at elevated temperature.
2. Determination of swelling pressure of compacted bentonite (dry density = 1.6 Mg/m^3) at temperatures of 25, 40, 50, 70, and 90^0 C.
3. To conduct compressibility test of bentonite and local soil at temperatures of 25, 50, 70 and 90^0 C.

1.3 Thesis Overview

The whole thesis divided into five chapters.

Chapter 1 mainly focusses on the background and motivation from the earlier work, primary objectives of the work, and overview of the project work.

Chapter 2 presents a detailed literature review of the work undertaken. The chapter covers introduction, structural unit of clay minerals, structure of compacted bentonite, forces, and charges in the clay system, particle association of clay i.e. orientation of fabric presented in detailed. The chapter also presents a review of swelling mechanism in clay, swelling pressure, and it's testing in the laboratory, swelling pressure testing devices available and the experimental setup to carry out thermal and thermo-hydraulic tests briefly reviewed. Compressibility behavior of bentonite and effect of higher temperature on swelling and compressibility of clay soils also discussed.

Chapter 3 mainly presents the test to conduct the physical properties of the clays used in this study and the methods adopted for determining the properties of the clays. The physical properties determined include specific gravity, particle size distribution, minerals composition using XRD and SEM analysis, natural moisture content, consistency limits, linear shrinkage, differential free swell and determination of specific surface areas. The chapter also presents the schematic diagram of the developed modified oedometer its calibration chart and the experimental setup for swelling pressure and consolidation test, also cover the procedure adopted for preparing saturated slurried specimens and compacted specimens of the clay.

Chapter 4 outlines the results and discussions coming from swelling pressure test and consolidation test at various temperature. Also, cover water content and degree of saturation variation with temperature for both the bentonite and local soil.

Chapter 5 covers the main conclusions drawn based on the findings of this study and the scope of further study.

Chapter 2

Literature Review

2.1 Introduction

At the time of construction and long-term process of a repository, compacted bentonite can work as a barrier, protect the waste canister and restrict the transfer of radionuclide coming from the waste after the possible failure of the canister (Wersin et al., 2007). Bentonite is a type of clay mineral mainly composed of montmorillonite in a smectite group. Montmorillonite is a swelling mineral. Other minerals which also found in bentonite are non-swelling minerals such as quartz, feldspars, micas, carbonate and sand. Nath and Dalal (2004) has done an experiment on physical properties of soil and found that plasticity index of the soil increases with the increase in liquid limit of soil. Clayey soils pose many problems to Geotechnical Engineering structures due to their large scale volume changes and poor shear strength. Moreover, clayey soils have more ability to swell than other soils.

Soils with high swell potential are called expansive soils. These soil types change in size as a result of variation in moisture content. Normally, expansive soils increase in size and swell when they absorb water and reduce in size and shrink when they become dry. Volume change in soil leads to distortions in the form of settlement due to contraction as a result of dryness or in the form of expansion due to swelling as a result of the absorption of water and increased humidity. The damage caused by expansive soils is very significant because of their high volume change which comes from swell-shrink behavior. At high plasticity index, the probability of shrinkage and swelling increases. There is a various application where these expansive soils have to be engineered in such a way so that it can achieve the desired behavior as a barrier for landfill liner and back-filling materials for nuclear waste disposal system (Yong et al., 1986).

Change in temperature alter almost all the geotechnical properties of soils. Increase in temperature increases the thickness of diffuse double layer. But, the dielectric constant of pore fluid decreases. These two effects have opposite influence on the properties of soils, but the effect due to the former dominates. Also the moisture content of soil increases

leading to decrease in shear strength of soil. Increased temperature leads to dispersed structures. Temperature also affects the chemical and biological processes occurring in soil. Adsorption and transport of water are affected by temperature. Constant attempts have been made by various researchers to understand the mechanisms which control soil behaviour. Such research efforts show that there are primarily two mechanisms which govern the behaviour of fine grained soil, namely: (i) shearing resistance at particle level (Mechanism-I) and (ii) double layer forces (attractive and repulsive forces) (Mechanism-II).

The parameters that influence mechanism-I are: particle roughness, size and shape, density, water content, applied stress and for mechanism-II are: bounding forces, valency of ions hydration of ions, pH, temperature, di-electric constant. More often, mechanism-I operates at low void ratio and low water content and mechanism-II operates at high void ratio and high water content. The behaviour of coarse grained material is primarily controlled by the mechanism-I. Both mechanisms I and II independently or simultaneously operate in fine grained soils, depending upon water content, void ratio and stress history. For example, fine grained soil at low water content and low void ratio, the mechanism-II will prevail. However, the two mechanisms primarily depend on the 'net attractive and repulsive forces' in soils. In general, theories developed for a pure system may not be of direct relevance to obtain the realistic picture of the soil behaviour.

The volume change behaviour of fine grained soils is essentially a function of the predominant type of clay mineral present in the soil system. It is well recognized that soils rich in smectite group of clays undergo a large volume change, while soils with fixed lattice such as kaolinite, muscovite and illite clays undergo relatively less compressibility (Mitchell 1993). The volume change behavior of compacted bentonite under different temperature was studied by Romero (1999) and Sultan et al. (2002). The study shows that temperature produces opposite effects to the soil such as swelling or retraction, softening or stiffening, a decrease of the elastic domain or over consolidation i.e. it can be reversible or irreversible. Effect of temperature cause a contraction in the volume of normally consolidated soil, whereas expansion in volume of over-consolidated soils (Baldi et al., 1988). The maximum swelling pressure of the sample achieved when the degree of saturation equal or more than 95% for the dry densities (1.29-1.95 g/cm3) at a temperature 20-80 °C (Shirazi et al., 2008).

Important properties of clay soil such as low permeability, less porous structure, and plasticity to act as a barrier lost due to the effect of temperature, which may develop from the waste. These buffer material in compacted condition played an important role to prevent the waste at its initial stage. At this stage radioactivity and the temperatures levels due to the production of the heat of the stored waste very high. This time, available information related to the effect of temperature on swelling characteristics of compacted clay soil is limited. Therefore, it is s a challenge to scientists and researchers working in this field to understand the temperature effect on swelling pressure of compacted clay thoroughly.

2.2 Identification of expansive soil

An expansive soil can be identified by the potential of the soil to swell independently of field conditions such as water content and surcharge pressure. The potential for swell depends on many factors: 1) amount and type of clay mineral; 2) soil structure, such as particle arrangement, bonding, and fissures; and 3) nature of the pore fluid and exchangeable cations. According to Chen (1975), expansive soils can be identified by three methods. The first method is mineralogical identification, the second one is indirect measurements, such as the index property, including Atterberg limits tests, linear shrinkage tests, free swell tests, and activity method and the third one is a direct measurement in the laboratory. Federal Housing Administration (1973), classified the expansive soils based on their plasticity index and liquid limit value as shown in Table 2.1. The Department of Army (1983) also classified the potential of swell according to their Atterberg limits as shown in Table 2.2.

Table 2.1 Classification of expansive soils (Federal Housing Administration, 1973)

Classification	Plasticity index (%)	Liquid limit (%)	Soil group
Non-expansive	0-6	0-25	A
Marginal	6-10	25-30	B
Moderately expansive	10-25	30-50	C
Highly expansive	>25	>50	D

Table 2.2 Classification of swelling soils (Department of Army, 1983)

Classification	Plasticity index (%)	Liquid limit (%)	Potential swell (%)
Low	<25	<50	<0.5
Marginal	25-35	50-60	0.5-1.5
High	>35	>60	>1.5

Altmeyer (1955), suggested a classification method of expansive soil to determine the potential of swell according to their shrinkage limits (%) and linear shrinkage (%) value as shown in Table 2.3.

Table 2.3 Soil expansivity prediction by shrinkage limits and linear shrinkage (Altmeyer, 195

Degree of expansion	Shrinkage limit (%)	Linear shrinkage (%)
Critical	<10	>8
Marginal	10-12	5-8
Non-critical	>12	<5

2.3 Structure of clay mineral

The mineral composition of the expansive soil includes quartz, and smectites as major components and feldspar, calcite, chlorite minerals as minor components. Primary minerals (e.g. quartz and feldspar) having dimensions larger than the secondary clay (kaolinite, illite, and montmorillonite). Because of the smaller in size, secondary minerals having a very high specific surface area (m^2/g) than primary minerals. These clay minerals are available in the crystalline form of relatively complicated structure. These minerals divided into three groups according to their crystalline arrangement, and it found that all the three groups have roughly similar Engineering properties. Although mineral structure complicated, mineralogical investigation of different clay mineral demonstrated that they made of two basic building blocks silica tetrahedral and alumina octahedral.

The silica tetrahedral consists of a central silicon atom surrounded by four oxygen atom arranged as the apex of the equilateral triangle. A number of tetrahedral combines to form a sheet of silicon and called as silica sheet. The alumina octahedral consists of a central alumina atom surrounded by six hydroxyl ion. A number of octahedral combines to form a sheet of alumina and called as gibbsite sheet. These sheets stacked together and formed a crystal. In the case when it is air dry, bonding between these layers possible by van der Waals and cation exchange process. As above mentioned types of bonding are weak, and they broke very easily when any polar liquid enter between the layers (Mitchell, 1993). The particular arrangement and chemical compositions of silica tetrahedral sheets and alumina octahedral sheets determines the type of clay mineral and its general characteristics. The structural arrangement of clay mineral families shown in Figure 2.1.

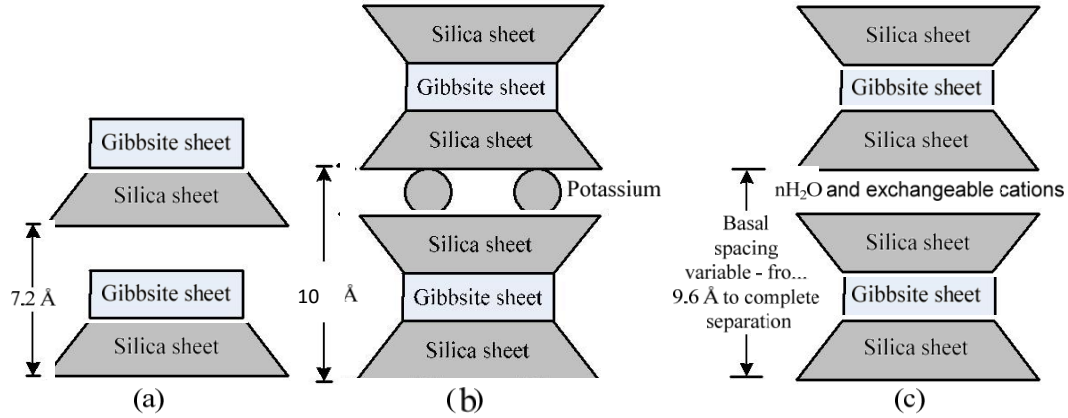


Figure 2.1 Clay minerals structural arrangement: (a) kaolinite, (b) illite and (c) montmorillonite (Mitchell, 1993).

2.3.1 Kaolinite

Sheet or plate-like structure in the shape of folded, twisted or rod. It consists of alternating sheets of single silica tetrahedral and single alumina octahedral produces 1:1 clay layer of kaolinite mineral. In between the layers both the sheets strongly bonded by hydrogen bonding (Yong and Warkentin, 1975). Because of the presence of hydrogen bond a strong bond, kaolinite crystal does not hydrate and therefore, a very stable mineral. It has very fewer chances of volume change when contact with water or moisture. This mineral contains no interlayer water since both the sheets fit together very closely. The specific surface area (SSA) of the mineral is about $15 m^2/g$.

2.3.2 Illite

It consists of two silica tetrahedral sheets with a central alumina octahedral sheet. Both the alumina and silica sheets share their oxygen atoms with each other, which result in the development of four oxygen thick layer. Potassium ions take the position between adjacent oxygen base planes. The potassium ions bind the two sheets together more firmly than that of montmorillonite with the result that the lattice is less susceptible to cleavage. Because of this property Illite, not expand its volume so much in the presence of water as that of montmorillonite though it expands more than kaolinite. In this type of mineral the negative charge to balance the potassium ions comes from the aluminum substitution within the octahedral layer. These mineral show properties between both the mineral kaolinite and montmorillonite. It has fewer chances of volume change when in contact with moisture or water. The specific surface area of the mineral is about $90 m^2/g$.

2.3.3 Montmorillonite

Bentonite, a smectite group of clay, contains a large quantity of mineral montmorillonite. This clay expands in volume when comes in contact with moisture. According to the modern concept, it is made of two silica sheet with a central alumina sheet (Mitchell and Soga, 2005). The structural unit forms a weak bond and it can break easily. All the tip of tetrahedral points in the same direction. The tetrahedral and octahedral combine so that tip of tetrahedral of each silica sheet and one of the hydroxyl layer of octahedral form a common layer. There is absolutely no bonding between two montmorillonite due to the presence of oxygen base layer. Montmorillonite and illite have similar composition. The outstanding feature of the mineral is that water and other, polar molecule can enter in the layer, and expand in the c-direction depending upon the polar molecules in amount present. Hence, it is of having very high CEC and specific surface area than the other two minerals discussed above. The specific surface area of the mineral is about 800 m^2/g.

2.4 Structure of compacted bentonite

As we know clay minerals consists of a lot of particles. Multiple pores exist inside the bentonite structure (Pusch, 1982; Delage et al., 2006). The micropores defined as

the pores lying inside the aggregates (intra-aggregate pores) (Delage et al., 2006). The macropores are the pores in between the aggregates (inter-aggregate pores). The pores between the particles are sometimes called as the interparticle pores or mesopores (Delage et al., 2006). The change in the microstructure of compacted bentonites is responsible for the change in hydraulic conductivity, swelling pressure and cation or anion diffusion capacities (Pusch, 2001).

Bradbury and Baeyens (2003), subdivided porosity as interlayer pores (size 0.2 to 1 nm), micropores (size less than 5 nm) and macropores (size 5 to 20 nm). The schematic diagram of the microstructure of powdered air-dry bentonite shown in Figure 2.2. The variety of possible soil fabrics and interparticle forces lead to the variety of pore structures (Mitchell, 1993). The unit layers, particles, and aggregates form various types of pores in clay soil. In compacted soils the clay particles combined, and hence there is a chance of an increase in the formation of the aggregate microstructure (Delage et al., 2006).

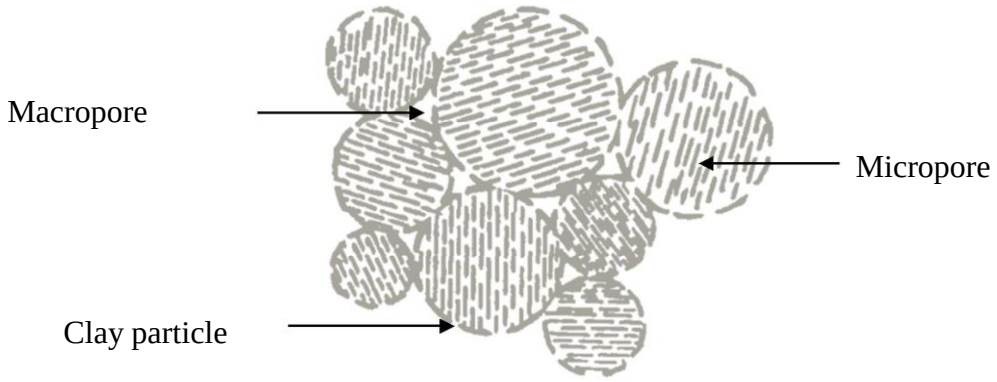


Figure 2.2 Schematic diagram of the microstructure of powdered air dry bentonite (Pusch, 1982).

Net attraction of soil system results in flocculated particles and net repulsion results in dispersed structure. Inter-particle forces (attractive as well as repulsive) was observed to have a major effect on engineering properties of soil, such as Atterberg's limit, swelling and compressibility characteristic of clay soils (Mitchell, 1993). In addition to both the inter-particle forces, the fabric of soil also plays an important role. Fabric is nothing but the particles arrangement, particle group and pore spaces in soil.

2.5 Forces and charges in clay system

As we know clay particles, they carry a charge on its surface which is negative in nature. The charge developed because of both the isomorphous substitution and a breakage of the structure at its edges. When it is in a dry condition, the charge on the surface is balanced by some of the exchangeable cations like Na^{+}, Ca^{2+}, Mg^{2+}, and K^{+} neighboring the clay particle. When the water added to the soil, the cations mentioned above, and some of the anions, they float around the clay particles, and the process is called diffused double layer (DDL) which is the result of clay water electrolyte interaction. On the negatively charged surface of the clay, cations held strongly. These cations termed as adsorbed cations. The adsorbed cations try to diffuse away from the surface of clay, and it tries to equalize the concentration throughout pore water. The diffusion tendency of adsorbed cations and electrostatic attraction together would result in cation distribution neighboring to each clay particle in suspension. Figure 2.3 shows such ions distribution neighboring to a single clay particle. Close to the surface, there is a high concentration of ions which decrease outwards. Thus, there are double layers of ions compressed layer and diffuse layer and hence the name double layer.

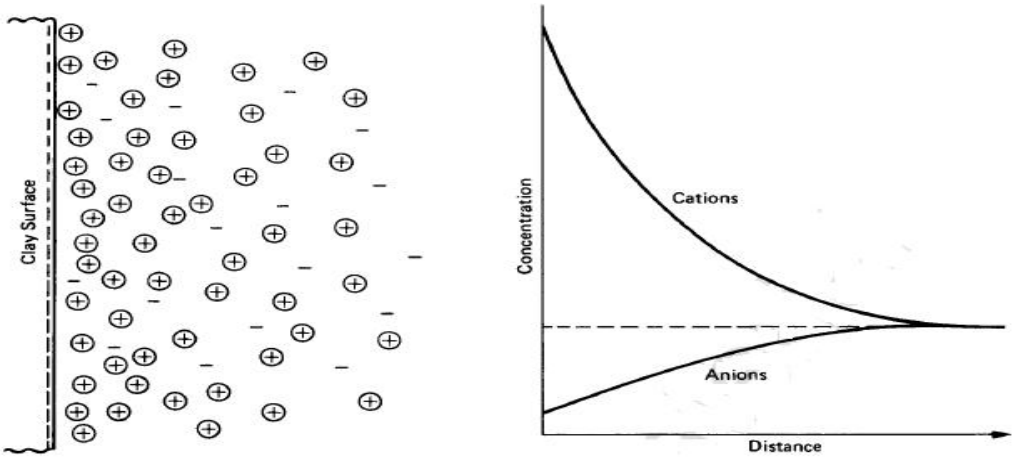


Figure 2.3 Ion distribution neighboring to a clay surface using DDL concept (Mitchell and Soga, 2005).

Increase in ionic valency for a given concentration decreases the double layer distance and there by aggregation of particles occur. Bolt (1956) found that for sodium montmorillonite, the void ratio change for a pressure of 0.1 to 1.0 kg/cm2 was 1.5 times higher than that of calcium montmorillonite. In general, as the valency increases, the

permeability increases, whereas, compressibility, swelling and plasticity characteristics decrease.

In the case of soils containing the smectite group of clays, an increase in the pore electrolyte concentration, decreases the plasticity, swelling and compressibility characteristic of the soils. Mesri and Olson (1971) noted that in the case of sodium montmorillonite, increasing the electrolyte (sodium chloride) concentration from $5x10^{9}$ to $1x10^{11}$ N, the void ratio decreased from 11.18 to 5.4 for an effective consolidation pressure of 0.05 kg/cm2, while, in the case of calcium montmorillonite, an increase in the electrolyte (calcium chloride) concentration from $1x10^{3}$ to $1x10^{7}$ N resulted in the void ratio decreasing from 2.18 to 1.84 only, at the same pressure of 0.05 kg/cm2.

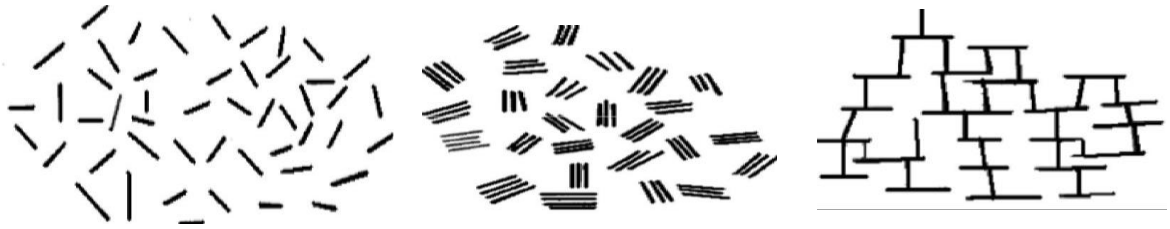
Increase in the dielectric constant, increases plasticity, swelling, compressibility and decreases shear strength characteristic of montmorillonite soils, while there will be opposite effect on kaolinite soils**.** Mesri and Olson (1971) have presented data on consolidation curves for montmorillonite with different organic fluids with varying dielectric constants, and have shown that the void ratio decreases, with the decrease in dielectric constant**.**

2.6 Particle association in clay

In a clay soil, a basic idea to understand how the soil fabrics formed and how they changed with time and loading throughout the history of that clay is very important. van Olphen (1977) gives the classification as 1) face to face (FF), 2) edge to face (EF), and 3) edge to edge (EE). The second and third classification produce flocculation of viscous gel while the first classification produces less viscous oriented aggregate. Dispersion is used to describe the disassociation of flocculated particles. Here flocculated and aggregated terms used to assemble the multi-particle, whereas, deflocculated and dispersed used to single particle or particle group acting independently.

An understanding of the different factors which is responsible for various association shown in Figure 2.4. With the help of a type of association viscosity or gel forming tendencies of clay can be determined, which depends on the clay concentration. Microscopic study of the clay soil by different researchers shows that inside the clay aggregates, the clay

fabric oriented mostly in a face to face arrangement (Saiyouri et al., 2000). Clay minerals consist of several unit layers stacked on top of one another to form a particle. A montmorillonite particle is usually made of several unit layers (Saiyouri et al., 2004). The variety of possible soil fabrics and interparticle forces lead to the variety of pore structures (Mitchell, 1993). Understanding structure and fabric of clays are important for understanding their engineering behavior. Delage et al. (2006) have shown that the behavior of clays at microscopic level plays a significant role on the volume change behavior at the macro level. Tripathy et al., (2010) also found the same result for the physicochemical interactions between clay particles and pore fluid due to an increase in suction and normal stress.



(a) **(b)** **(c)**

Figure 2.4 Modes of particles association (a) Dispersed and deflocculated (b) aggregated but deflocculated (c) flocculated but dispersed (van Olphen, 1977)

2.7 Swelling Mechanism in clay

The swelling behavior of expansive clays exposed to water produced because of two mechanisms one is crystalline swelling and another one is osmotic swelling (Madsen and Müller-Vonmoos, 1989). It produced when expansive clay mineral is allowed to saturate under controlled volume conditions. Both mechanisms influenced by the breakup of montmorillonite particles and by the demixing of cations. According to Yong (1999), the difference between this two mechanism crystalline and the osmotic swelling is only due to hydration structure of the water.

The first mechanism developed because of the hydration of ions when montmorillonite absorbs water is nothing but crystalline swelling. Madsen and Müller-Vonmoos (1989), found that in the case of unconfined condition montmorillonite volume

may double its original volume while in the case of confined condition it can be more than 100 MPa only due to crystalline swelling. When the swelling of the mineral is restricted, the crystalline swelling pressure of very high degree will develop, reaching several thousands of kPa. The second mechanism, which appears when montmorillonite absorbs water i.e. change in volume of the clay mineral beyond crystalline swelling is due to osmotic phenomena. It occurs because of the interactions between diffuse double layers and the Vander Waals attraction. It is also called as double layer swelling.

2.8 Swelling pressure

The pressure required to keep a constant void ratio of the specimens during the hydration process of unsaturated expansive soil is called swelling pressure (Sridharan et al., 1986). When bentonite absorbs water under restrained boundary condition, they exhibit swelling pressures. Insertion of water molecules within the interlayer and interparticle pores causes montmorillonite clay to swell (van Olphen, 1962). However, if volume change is not allowed, the specimen will exert swelling pressure equivalent to the net repulsive force exerted between the clay platelets (Bolt, 1956; Tripathy et al., 2004). The magnitude of swelling pressure depends on the specific surface area, available exchangeable cations, temperature, initial dry density, and initial water content (Villar, 1999; Tripathy and Schanz, 2007).

Swelling pressure behavior of bentonite in compacted form can explain using the concept of before, during and after water uptake. From the Figure 2.5, it can explain that before water uptake, bentonite it is a mixture of montmorillonite, voids, and other minerals having non-swelling properties. At the time of water uptake, montmorillonite absorbs water and into interlayers and swells occupying the void in the bentonite. Therefore, montmorillonite increases its volume, and the swelling pressure occurs. After water uptake, there will be no voids available to absorb water so the bentonite volume cannot increase. At this stage the soil will be in equilibrium condition and hence the swelling pressure can measure.

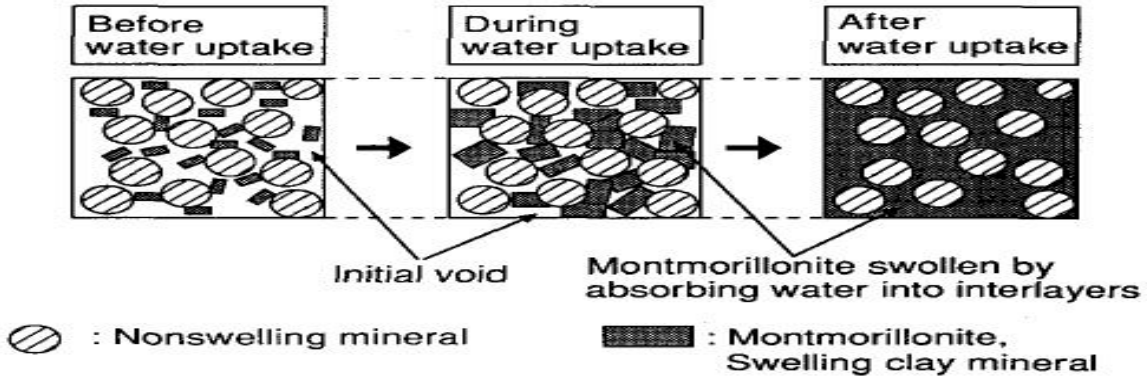


Figure 2.5 Mechanism of swelling pressure on compacted bentonite (Komine and Ogata, 1994).

2.8.1 Swelling pressure testing in Laboratory

Measurements of swelling pressure of expansive soils during the hydration process have been conducted in the laboratory by different methods (Sridharan et al., 1986; Komine and Ogata, 1994; Villar 1999; Tripathy et al., 2004). These methods are constant volume (i.e. isochoric condition), swell-underload, and swell-load (Sridharan, et al., 1986). All three gives a different value of swelling pressure. In swell-load, the specimen is inundated and allowed to swell under seating load. Loads added after an equilibrium condition reached by the soil sample. In swell-under load, several identical specimens are allowed to swell or to compress under different vertical loads to reach equilibrium conditions.

The swelling pressure defined as the pressure required to bring the soil specimen to its initial void ratio. In the case of constant volume method (no swell method), the volume of specimen is maintained constant by applying pressure until there is neither swelling nor compression occurred during inundation. That is why this method is also known as strain controlled test. Various methods of swelling pressure measurement shown in Figure 2.6. It observes that swell load test gives the highest value, constant volume test gives intermediate value, and swell under load test gives the least value of swelling pressure (Sridharan et al., 1986).

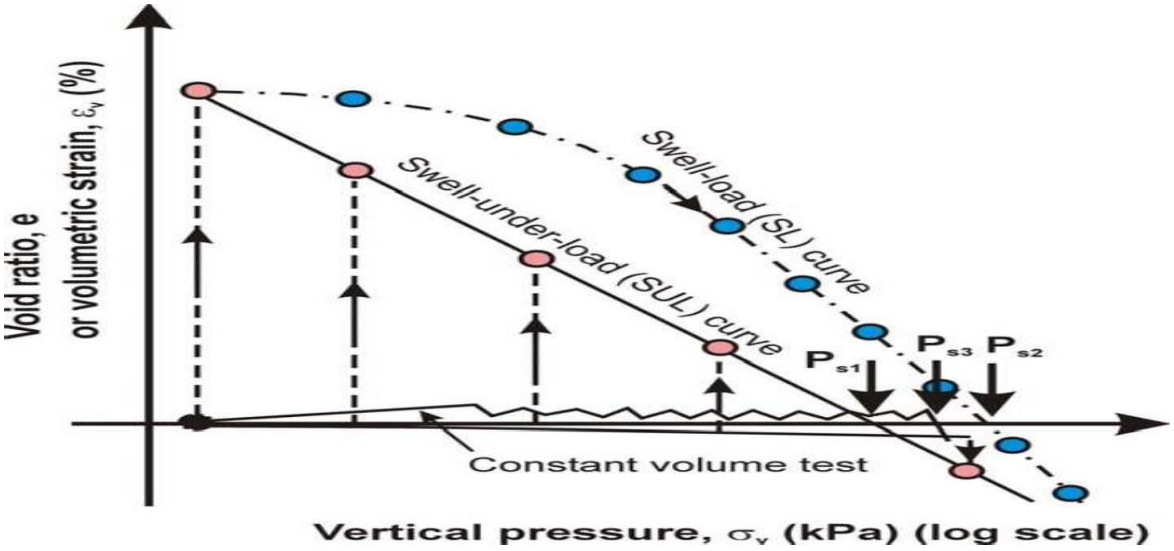


Figure 2.6 Different methods of measurement of swelling pressure (Sridharan et al., 1986).

In case of constant volume method, swelling pressure of bentonite in compacted form can determined by saturating the specimen with distilled water in one step; namely one step constant volume swelling pressure test. It can also determined by applying suction using axis-translation technique (ATT) and vapor equilibrium technique (VET); namely multi-step constant volume swelling pressure test. Various techniques have adopted for saturating the specimen in one step swelling pressure test. Bucher and Spiegel (1984) used high water pressure to saturate the specimen in one step method. The water was supplied using a liquid pump which can deliver high liquid pressure to the specimen. Sridharan et al. (1986) used water flooding method to saturate the specimen. The test performed on an oedometer apparatus. The specimen was allowed to swell in distilled water. The load where no further swelling observed is nothing but swelling pressure. Swelling pressure measurement conducted on specimen equilibrated at various applied suction is known as multistep swelling pressure test (Ariffin and Schanz, 2007). Multistep swelling pressure tests are conducted using suction controlled oedometer (Romero, 1999; Villar, 1999; Villar and Lloret, 2004).

2.8.2 Swelling pressure measurement devices

Madsen and Muller-vonmoos (1985) used a modified oedometer to measure the swelling pressure of clay soil. They mainly use this device to measure swelling pressure of compacted clay for the case when the clay soil subjected to a higher temperature. The device

with the full experimental setup shown in Figure 2.7. They monitored the deflection of the specimen using two dial gauges. The swelling pressure measured by accommodating a pressure cell in between a loading plunger and a restrained bar.

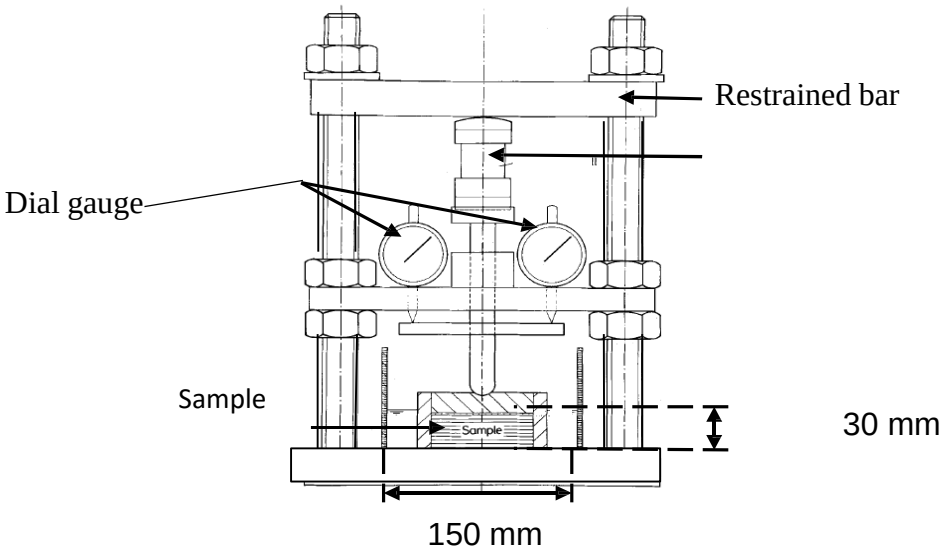


Figure 2.7 Setup for measuring the swelling pressure (Madsen and Muller-vonmoos, 1985).

For testing the swelling pressure of bentonite soil at high temperature, Bucher, and Müller-Vonmoos (1989) used the apparatus as shown in Figure 2.8. In the test, the water was supplied using the liquid pressure of 1000 kPa. The swelling pressure recorded through pressure transducer. The thermal element was mounted alongside the specimen and controlled by the temperature controller. The cell deformation was measured using dial gauge placed in the top of the apparatus.

Romero (1999) developed a device for measuring swelling pressures of compacted bentonites by constant volume method shown in Figure 2.9. The device facilitated with a load cell for measuring the swelling pressure. Clay specimens hydrated by circulating fluid from both the top and the bottom of the clay specimens. Several researchers have used the modified oedometer (Romero, 1999) for measuring the swelling pressure of compacted bentonites under constant volume condition (Schanz and Tripathy, 2009). A limited number of devices are available to measure the swelling pressures at elevated temperature. To conduct the swelling pressure tests at elevated temperature, a special arrangement, and modification of the existing devices required.

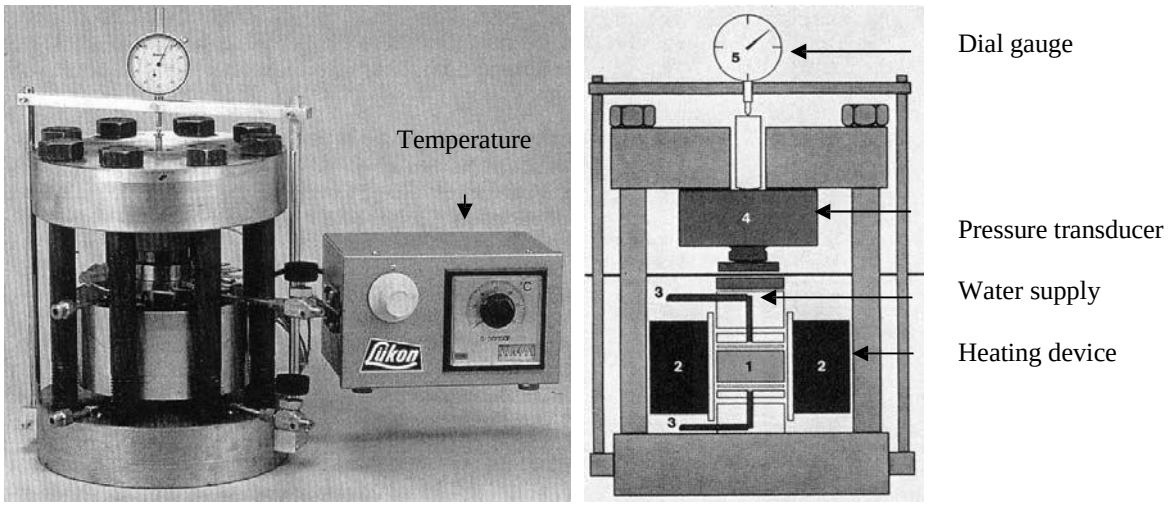


Figure 2.8 Swelling pressure test apparatus for high temperature (Bucher and Müller-Vonmoos, 1989) (a) experimental setup, and (b) cross section of the apparatus.

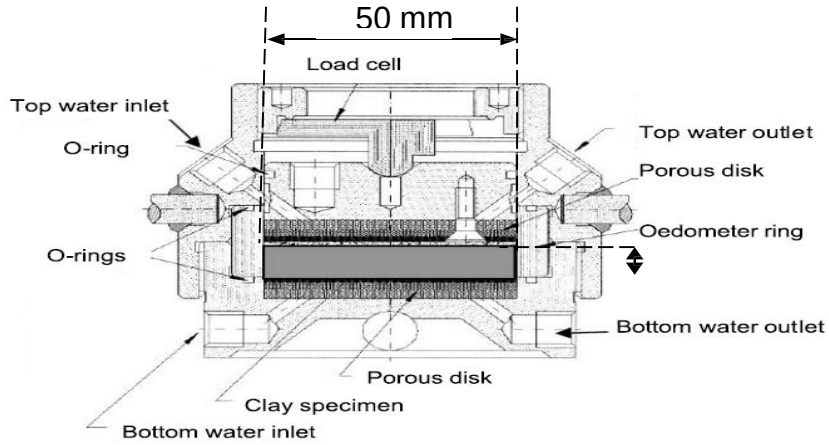


Figure 2.9 Schematic diagram of devices used for measuring the swelling pressure by constant volume method (Romero 1999).

2.9 Compressibility behavior of bentonite

Compacted bentonite used as a barrier in a nuclear waste disposal repository experience very high overburden pressure. Tripathy and Schanz (2007) found that in this condition, the pressure void ratio relationship of the soil is the main criteria for the stress convergence of the host rock. The compressibility behavior of expansive clay has done by several researchers, covering wide void ratios range with large pressure changes (Bolt, 1956; Sridharan et al., 1986). The compressibility behavior of clay soil can measure just by

saturate it initially with the water contents higher than its liquid limit. The saturated condition of clay is considered as a reference condition here to study the behavior of clays. Sridharan et al. (1986) have applied large pressure in the range of 6.25 to 300 kPa. Tripathy and Schanz (2007) stated that using diffuse double layer (DDL) theory, having a void ratio of bentonite at low pressures used, for the prediction of pressure void ratio relationship of bentonite at very high pressures. Baille et al. (2010) also work done on the consolidation of expansive clay at high pressures. The clay specimens were compacted before testing and were allowed to swell after saturation.

2.10 Effect of higher temperature on swelling and compressibility

Pusch (1980) measured the swelling pressures of compacted saturated montmorillonite at temperatures of 20 °C and 90 °C by hydrating the specimens with distilled water and different electrolytes. They reported significantly lower swelling pressure when tested at a temperature of 90°C than the temperature at 20 °C and reported that swelling pressure reduction was because of availability of less stable interlayer and inter-particle water at high temperature. Pusch et al. (1990) noted that temperature increase results decrease in swelling pressure of Ca-bentonite. Similar results were reported by Villar and Lloret (2004) for FEBEX bentonite (calcium and magnesium bentonite). Karnland et al. 1994 done experimentation on swelling pressure of saponite bentonite and found that as temperature increases swelling pressure also increases. The similar result reported by Pusch et al. (1990) on Na-bentonite. The same authors reported that the effect of temperature on the swelling capacity decreased with the increase in the confining stress. They also found that temperature is not important only for the development of swelling pressure of soil at very higher density, but it is also important at lower density. Romero et al. (2003) reported the temperature effects on multi-step swelling pressure test for compacted Boom clay. They found that the swelling pressure values at 22°C were higher than those at 80°C. In general, an increase in the temperature results in decreasing the swelling pressures of compacted bentonites. Further investigation is required to evaluate temperature effect on swelling pressure of bentonite soil.

The effects of temperature on the mechanical properties observed are often contradictory. Nevertheless, it can be concluded that the temperature effect on mechanical

properties is quite small. Cui et al. (2000) described two phenomena produced during heating: (1) expansion of soil constituents (solid and water): (2) mechanical weakening of the contacts between soil aggregates. Fang and Daniels, 2006 measured the volume change and compressibility of soil at a different temperature. They reported that a) with the increase in temperature compressibility of soil increases, because of this compression index Cc value increased radically and b) increase in temperature results instant volume changes and the magnitude of volume change depends on the magnitude of the change in temperature. In the case of compacted FEBEX bentonite, Romero et al. (2005) observed that the stiffness upon loading increased with temperature. On the other hand, with compacted Boom clay, Romero et al. (2003) found that the compression index is larger at higher temperatures. Saix et al. (2000) observed a contraction in volume during heating of the clayey soil under constant stress at 42, 160 and 800 kPa in oedometer.

Chapter 3

Material and Methods

3.1 Properties of Soil studied

This chapter describes only the laboratory methods commonly used to determine physical properties namely particle size distribution, hygroscopic water content, consistency limits, mineral compositions, specific gravity, and specific surface area of the clays. The soils used in this study are commercial bentonite clay procured from Bikaner Rajasthan and Rourkela local soil from Rourkela Orissa India. In this project I have used distilled water for all the test to found a better reference condition. Bentonite in dry powder forms chosen for this study. The bentonite powder was provided with an initial water content of 15% and was stocked in a plastic bag to maintain the water content constant. Physical properties of soil presented in Table 3.1.

The particle size of fine-grained soils and clay-size fraction (i.e. < 0.002 mm in diameter) are usually determined by the settling rate of the particles in a liquid following Stoke's law (Lambe and Whitman, 1969). The most commonly used technique in geotechnical testing is the hydrometer method IS 2720 Part 4 (1985). About 50 grams of oven dried soil passing through 75-micron meter sieve size weighted correctly and put it into a beaker. Add 100 cc of fresh (2%) solution of dispersing agent into the soil. Add distilled water if necessary and soak the soil for few hours. Transfer the soaked soil with water into the dispersion cup up to two third full and stir the mixture for 15 minutes. Wash the soil mixture into standard measuring jar and make the total volume of soil suspension exactly 1000 cc by adding distilled water. Calibrate the hydrometer and determine the various hydrometer corrections. Shake the soil suspension thoroughly without any loss. Start the stopwatch. Insert the clean hydrometer slowly without allowing any oscillation in the suspension. From the test results presented in Figure 3.1, clay fraction found in bentonite soils and local soils are 71 % and 20 % respectively.

The liquid and plastic limit of the soil determined following methods described in IS 2720 Part 5 (1985). The shrinkage limit of the soil is also determined following methods described in IS 2720 Part 6 (1972). These limits found out in the laboratory to study the

plasticity property of the soil. In the case of bentonite soil, it is very difficult to calculate liquid limit in Casagrande apparatus, so cone penetrometer with 20 mm penetration is used. The plastic limits of the clay determined by rolling the standard thread of diameter 3mm before the beginning of crumble. Plasticity index which gives the idea of soil characterization using plastic nature is nothing but the difference in the liquid and plastic limit of the soil. In the measurement of shrinkage limit of soil, clay specimens were prepared at 1.2 times their respective liquid limit values and placed within a greased shrinkage dish. Mass measurements were frequently conducted until no further reductions were noted. A typical value of liquid limit for montmorillonite is within the range of 100 to 900%, whereas the liquid limit for kaolinite is around 30 to 60% (Lambe and Whitman, 1969).

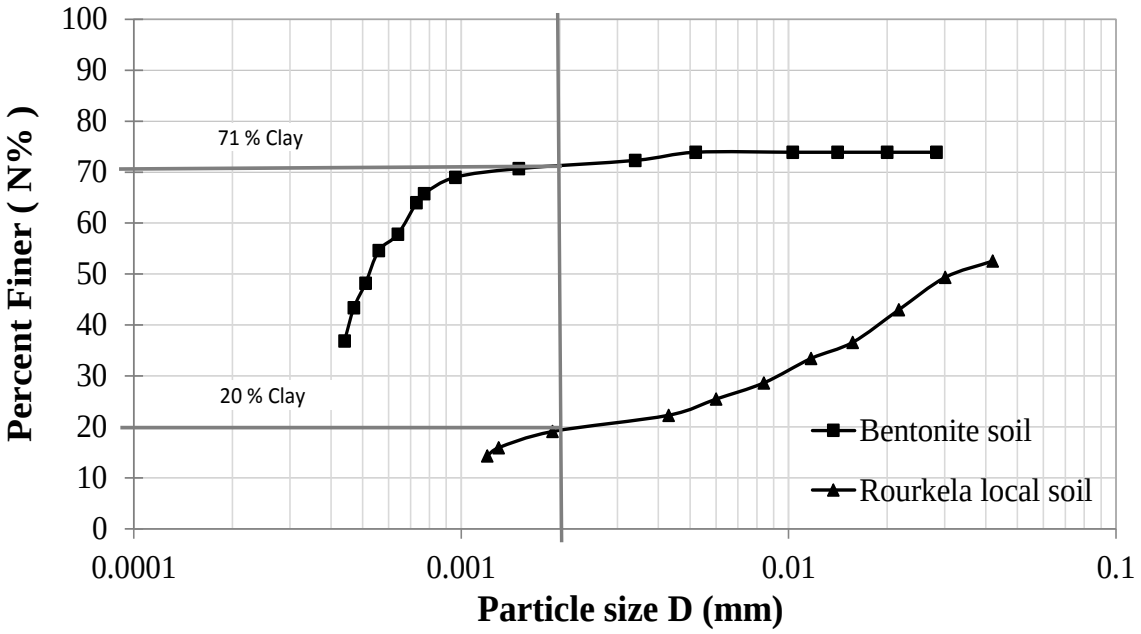


Figure 3.1 Grain size distribution curve of different soil studied

The hygroscopic water content also known as natural moisture content is the water film that adheres to the clay particle when it is in equilibrium with an atmosphere having a relative humidity of 50% at 20 °C and does not evaporate with ambient temperature. The amount of water adsorbed depends on the type of cations present and the specific surface area (Mitchell, 1993). The soils were oven-dried at a temperature of 105 °C described in IS 2720 Part 2 (1973) for 24 hours. The hygroscopic water contents were found to be 15% and 9 % for bentonite and local soil respectively. Linear shrinkage and differential free swell of

both the soil also determined following methods described in IS 2720 Part 20 (1992) and IS 2720 Part XL (1977).

The specific gravity of soil solids determined using the pycnometer (volumetric flask) method described in IS 2720 Part 3 (1980). The method can be difficult to conduct the specific gravity of highly swelling clays with high montmorillonite content. Bentonite, when mixed with water, will tend to flocculate. The incomplete penetration of water on the particles and possibility of chemical or physical change can affect the accuracy of measurements (Grim, 1968). For this reason, the specific gravity measurements for bentonites are conducted by using the non-polar liquid in place of water. In this study, kerosene was used as a substitute for water for determining the specific gravity of the bentonites, whereas deionized water used for the non-swelling local soil. The specific gravity values after correction were found to be 2.65 and 2.67 for bentonite and local soil respectively.

There are various methods available to measure the specific surface area of clays, either in dry or wet conditions. Determination of surface area in dry condition involves gas adsorption using Brunauer-Emmett-Teller (BET) isotherm method (Grim, 1968). Nitrogen gasses usually used for this purpose. The BET method only measures the external surface area of swelling clays, as the only limited amount of gas can adsorbed into the interlayer space (Alymore et al., 1969) whereas, in wet condition, adsorption and cation exchange take place thus both the internal and external surface areas measured. In wet condition total surface area of clay determine using Ethylene Glycol Monoethyl Ether (EGME) method (Mitchell, 1993; Cerato and Lutenegger, 2002). Specific surface area analyzer was used in this study to determine the external specific surface area of the clay using BET method. The external specific surface area of bentonite and local soil were found to be 70 m^2/g and 21.7 m^2/g, respectively. The external specific surface area for montmorillonite was reported to be in the range of 39.8 to 120 m^2 /g (Alymore et al., 1969), whereas the external specific surface area of kaolinite was reported to be about 16 m^2 /g (Grim, 1968).

The lower value of the specific surface area of both the soil found. The reason could be that the sample not dried properly, or the sample holder pipe not washed and dried properly, or the sample weight is taken more than the requirement. Using the BJH adsorption

method the pore volume and pore radius in a bentonite soil is 0.227 cc/g and 18.175 Å respectively while for local soil it is 0.064 cc/g and 18.142 Å respectively.

Table 3.1 Properties of soil studied

Property	Bentonite Soil	Local Soil
Liquid limit (%)	139	35
Plastic limit (%)	50	21
Plasticity index I_P (%)	89	14
Shrinkage limit (%)	38	15
Natural moisture content (%)	15	9
Linear shrinkage (%)	22	14
Differential free swell (%)	150	25
Specific gravity	2.65	2.67
Clay content (C %)	71	20
Activity = I_P / C %	1.17	0.7
Group symbol (IS Soil Classification System)	CH	CL
Specific Surface Area (using BET Method) (m^2/g)	70	21.7

3.1.1 XRD Analysis

Mineralogical compositions of clays are commonly investigated using X-ray diffraction analysis (Grim, 1968; Mitchell, 1993). The test is conducted using a Rigaku source and Philips power diffractometer with Cu Kα radiation at 35 kV and 40 mA. As per Bragg's law XRD mainly identify the mineral by relating the incidence angle of X-rays with its spacing of c-axis. Five gram of each of bentonite and local soil passing 75 microns with hygroscopic water contents tested. Now the soil is put into the sample holder diffractometer and fill it.

XRD pattern for both the soil obtained by scanning the soil over an angle 2θ ranging from 0 to 90^0 at a scanning rate of 5^0 / minute. The X-ray diffraction analysis of the bentonite soil and local soil shown in Figure 3.2 and Figure 3.3 respectively. The mineral found in bentonite soil using XRD analysis was montmorillonite, quartz, calcite, Alumina, Sodium erbium fluoride, fluorite, and hematite whereas, in local soil quartz, alumina, calcite, kaolinite, and hematite.

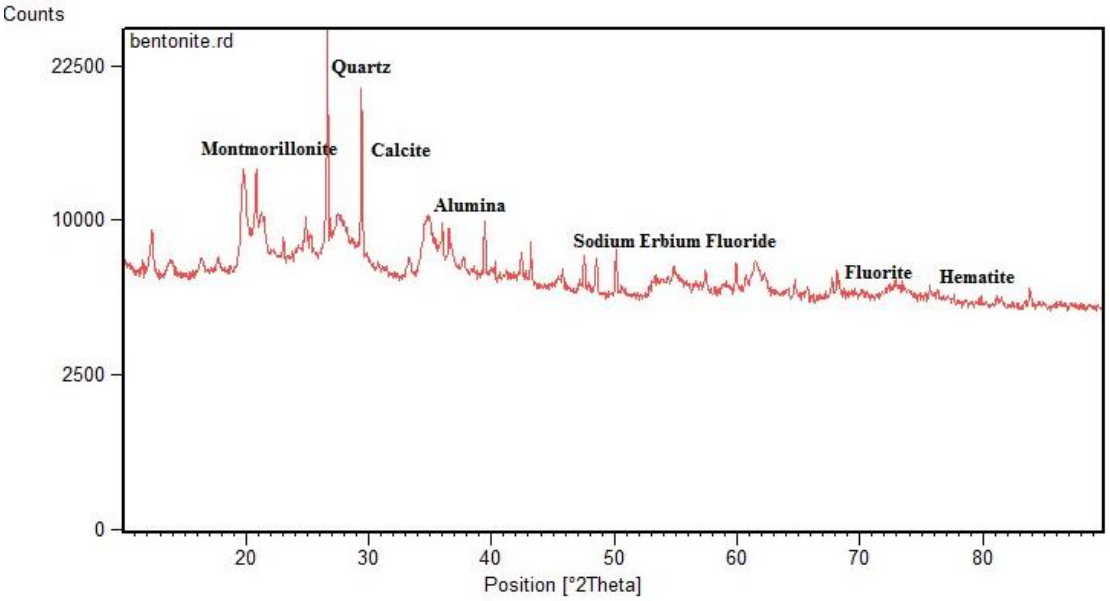


Figure 3.2 XRD Analysis of Bikaner bentonite soil.

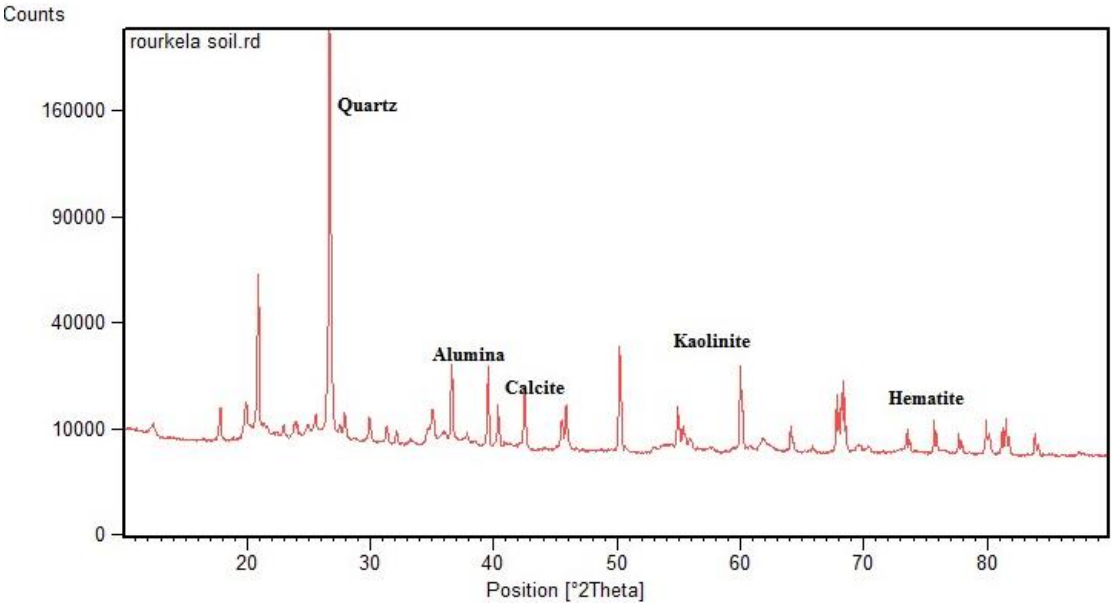
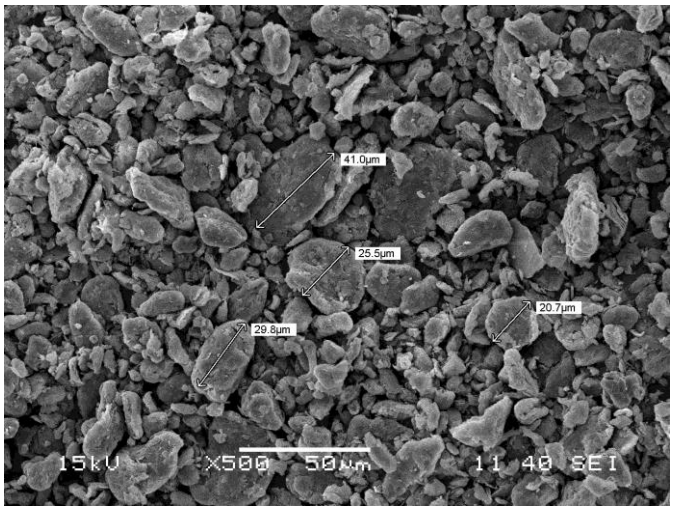


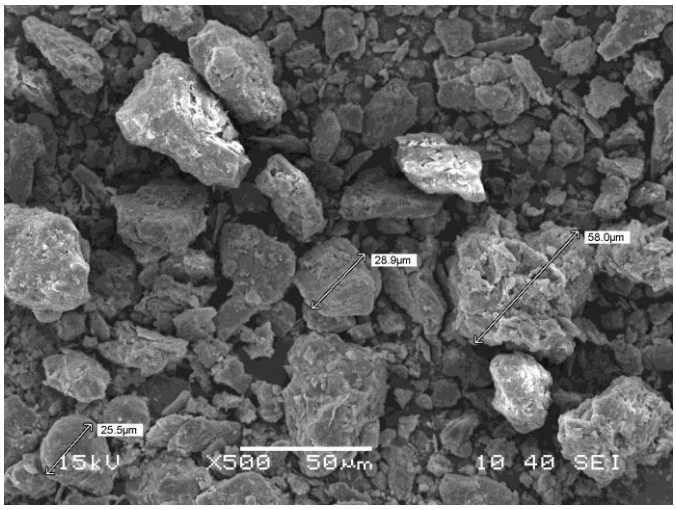
Figure 3.3 XRD Analysis of Rourkela local soil

3.1.2 SEM Analysis

SEM gives various features such as morphology (configuration) of samples, fabric (particle boundary relationships), and texture of the crystals. Below are Scanning Electron Microscope pictures of bentonite and local soil (Figure 3.4). From the microscopic picture of both the soil, it can conclude that bentonite soil is having a large amount of smaller size particle while in local soil it is less hence the overall voids will be higher in the case of bentonite soil so the specific surface area will be more.



(a) Bentonite soil



(b) Local soil

Figure 3.4 Scanning Electron Microscope picture of (a) bentonite soil and (b) local soil

3.2 Modified Oedometer

A new device modified oedometer developed for conducting swelling pressure and compressibility test at elevated temperature. In this device, two cells are used the outer cell and the inner cell. Outer cell welded to a mild steel base. An inner cell separates the inner chamber and oil reservoir. Specimen placed inside the inner cell, and a heating coil surrounds the specimen chamber of the swelling pressure device. To elevate the temperature of the soil sample, an electric coil used and connected to an external thermostat to perform the test swelling pressure at higher temperatures. To monitor the oil reservoir temperature a thermocouple used here. The temperature controller used in this study maintains the temperature of the heating element. The space between inside and outside ring filled with commercially available industrial grade oil. The oil reservoir is sealed at the top using a tight fit circular steel plate. Screws used for connecting the top plate to the outer casing. The inner cell holds a specimen assembly comprised of a specimen ring that enables accommodating a compacted bentonite specimen, two porous stone one at the top another one at the bottom of the specimen, and a locking collar with locking keys for holding the specimen ring in place (Tripathy et al., 2015). Two tubing connected at the base of the apparatus. Schematic diagram of the new product developed and the modified oedometer with the sample shown in Figure 3.5 and Figure 3.6 respectively. Temperature calibration graph shown in Figure 3.7 is used to calibrate the oil temperature as per the requirement of sample temperature since we need constant temperature for sample throughout the test.

The solid part of the device which consists of an inner chamber, outer cell, a cover plate for fixing at the top, and water inlet-outlet part completed with the help of Swadhin Enterprises, residing at Uditnagar, Rourkela. The electric coil, and thermocouple introduced in the device with the help of another manufacturer Process Instrumentation and Engineering Co. (PIECO) resisding at Koelnagar, Rourkela. The same manufacturer also manufactured temperature controller.

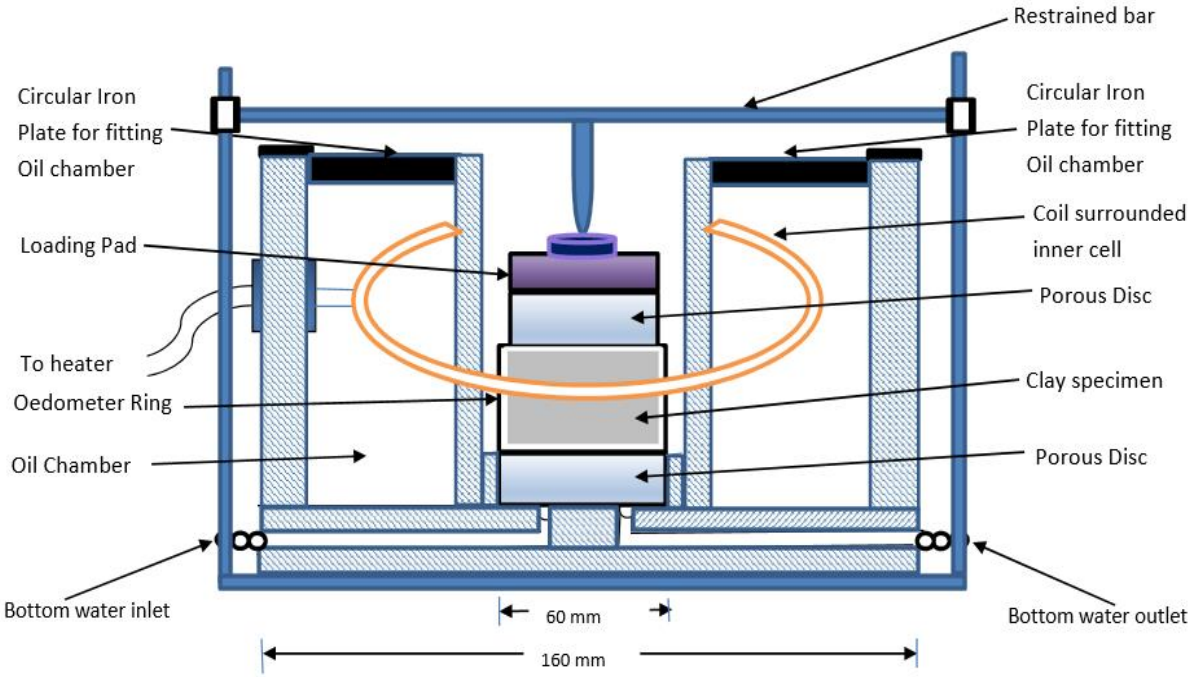


Figure 3.5 Schematic diagram of modified oedometer developed.

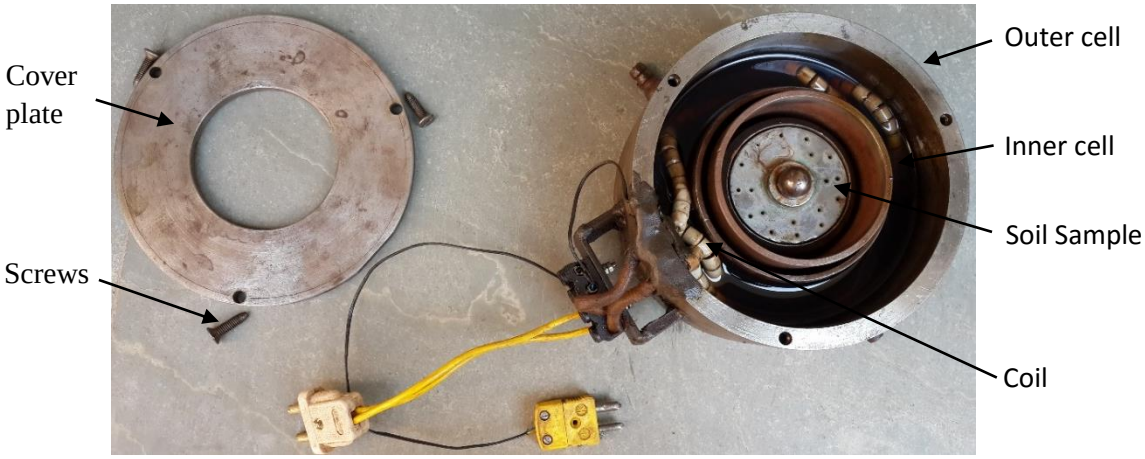


Figure 3.6 Modified oedometer with the specimen.

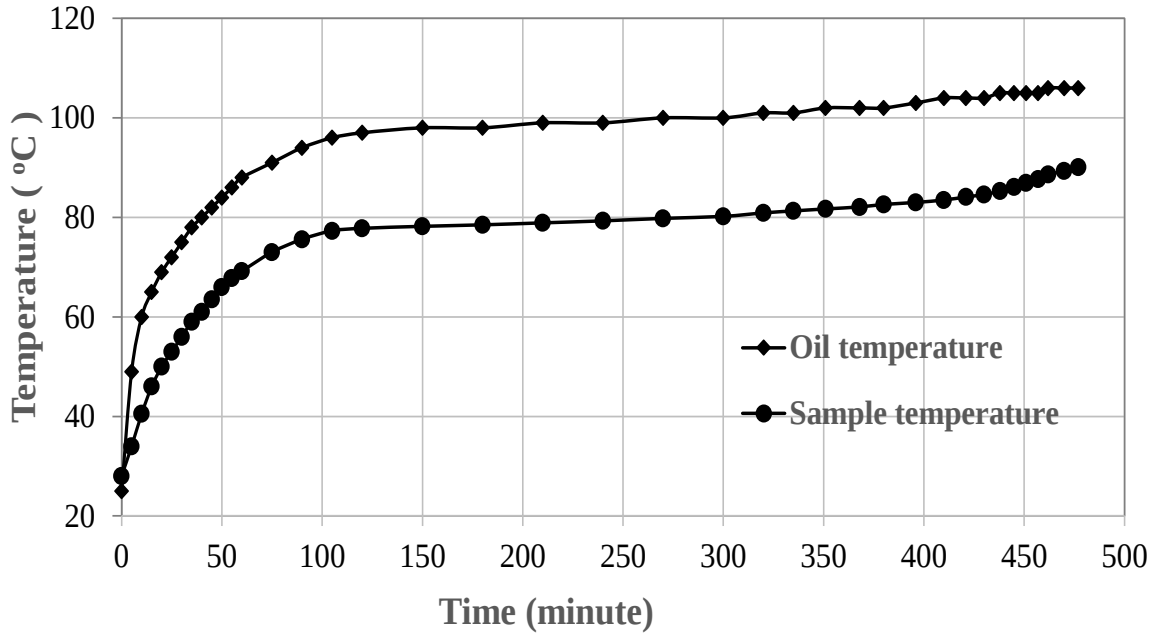


Figure 3.7 Temperature calibration graph for the modified oedometer

3.3 Swelling Pressure Test

3.3.1 Experimental setup

The experimental setup for the swelling pressure tests shown in Figure 3.8. It is composed of four parts: a testing cell (modified oedometer), a water reservoir to store water, a data logger, and a temperature controller. The testing cell contains a base made of steel, a specimen ring made of stainless steel, two porous stones, a loading plate also made of stainless steel, a top cover of mild steel, a pressure sensor and four screws for fixing all parts together. At the base of the mold inlet and outlet was provided for the supply of water. A load sensor kept on the sample for measuring the load value with the passage of time.

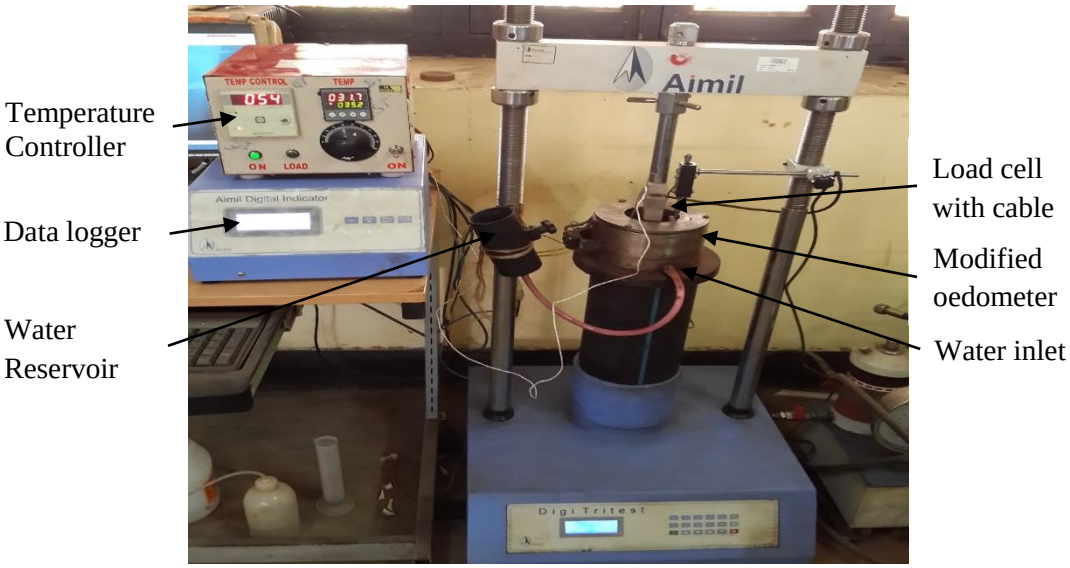


Figure 3.8 Experimental setup for constant volume swelling pressure test.

3.3.2 Constant volume swelling pressure tests

The swelling pressure of the soil specimen measures using constant volume method by digital strain meter. Distilled water supplied from the inlet via water reservoir and from the top using the laboratory wash bottle. The relationship between vertical swelling pressure and time elapsed from the time when water flow inside the sample started at different temperatures were plotted. At the end of the test water content and degree of saturation of the specimen measured and it was found that degree of saturation of the specimens very close to 95%.

In this test quantity of soil needed is calculated using volume and density concept. To achieve a target compacted dry density of 1.6 Mg/m^3 and cylindrical sample with a height of 20 mm and diameter 60 mm, a specially manufactured cylindrical column mold of 55 mm height and 60 mm diameter used. Bentonite in powder form having a natural moisture content of 15 % weighted (104 g) and put into the special mold consisting of cell ring at the bottom. Now compact the soil placed inside the mold at room temperature using static uniaxial compaction. The compressive force is applied statically at the top portion of the plug till the flange is in contact with the barrel of the mold. It can reach by applying vertical stress of 35 MPa and kept it for 1 hour. Just after the compaction is over compacted soil with the ring immediately put into the modified oedometer. Now installed the filter paper as well as porous plate each at top and bottom of the specimen surface.

The apparatus positioned in the loading frame and fixed it at top and bottom so that there will be no vertical as well as the horizontal displacement of the apparatus. Only soil will be allowed to swell or shrink. Connect the temperature controller with the apparatus and fix the value of oil chamber temperature as per calibration graph needed for the sample temperature. Wait till the sample temperature required is attained. Now set the load as well as deflection value initially zero. Supply distilled water from the inlet and the top. Allow the soil sample to swell till it reach an equilibrium value of swelling pressure. At the same time, a load cell which was installed in the load frame used for loading automatically record the value of the load, strain and time for water exchange of sample. Once the fluid supplied through the fluid inlets of the oedometer, the swelling pressure started to develop, and when it is completely saturated (assumed when water intake reach equilibrium value, Villar, and Lloret, 2004), it was considered that the test over.

3.4 Consolidation Test

3.4.1 Experimental setup

The experimental setup for the compressibility tests shown in Figure 3.9. It is composed of a testing cell (modified oedometer), two reservoirs for water storage, a temperature controller, and a dial gauge.

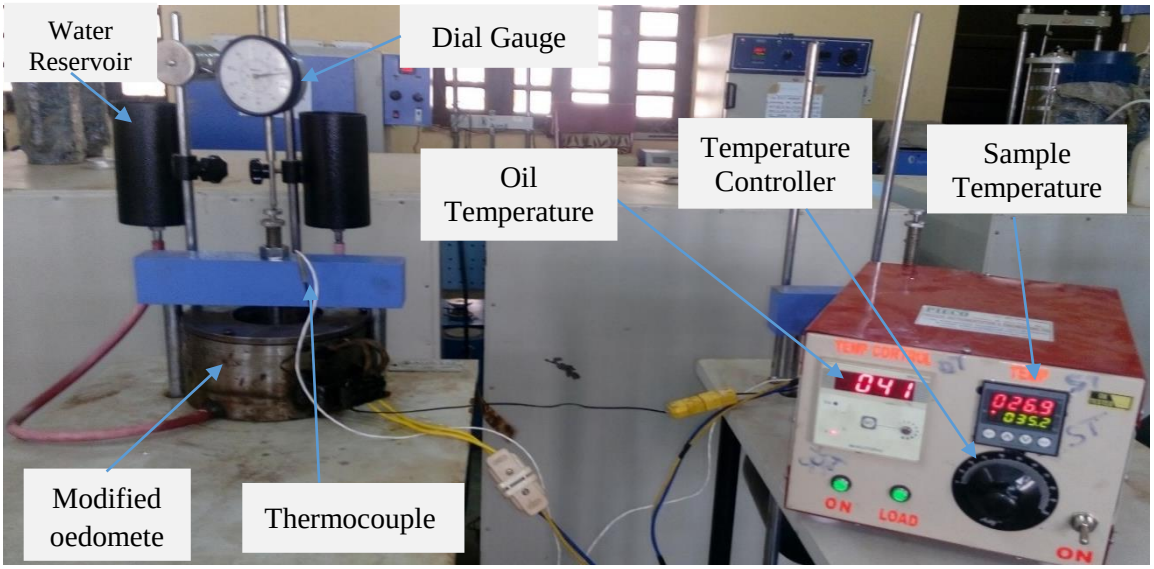


Figure 3.9 Experimental setup for consolidation of soil using modified oedometer

3.4.2 Methodology to Measure Compressibility

In the current study, a compressibility test is carried out on bentonite and Rourkela soil for the range of applied pressures between 54 kN/m^2 and 1727 kN/m^2. The test is carried out on initially saturated bentonite specimen following IS 2720 part 15 (1965) using a modified oedometer. The lever arm of the consolidation apparatus used for the current investigation is 16 cm. The diameter and height of the specimen ring used are 60 mm and 20 mm, respectively.

To measure the compressibility of the soil, it is mixed thoroughly with the amount of distilled water higher than 10-20 % of liquid limit (LL). At the time of mixing stir the soil with the help of trowel so that a homogeneous paste will form. Now the soil paste wrapped using a plastic bag and placed inside the desiccator and allow it to hydrate for 5-7 days before compaction for preconditioning and maturing. Later, the water content *w* of the sample is obtained. A modified oedometer, with a ring having dimension 60 mm x 20 mm, made of stainless steel used. Two Perspex square plate of size more than ring diameter used. Place the ring above one of the plate and use oil inside the ring surface. Put a small amount of the prepared soil sample in this ring and press it, at the same time remove the air void if available by tamping the ring over the plate. Repeat the same procedure till the ring is full of sample. Level the top portion of the sample by another one plate, if required press the sample from the top and add soil so that there will be no voids available. Now place the Filter paper on the top of the surface of the sample. Again place the plate on the top of the filter paper. Now turn the sample in such a way that upper surface will be bottom surface and vice versa. Level the other surface in the same way as it was done earlier, also place the filter paper above this surface.

The weight, diameter and height of the clay specimen were measured. At both the sample ends filter paper and porous stone were placed to distribute permeate water evenly from the whole end area of the sample. As the preparation of the sample in the ring over placed in the ring with the sample in the test cell. Now flow water into the cell and specimen kept for saturation. When the maximum swelling is over, then the loading is applied till there will be no change in the height. For each loading note the dial gauge reading and time. After full consolidation unloading the sample in the same sequence as done in loading and note the initial and final dial gauge reading with time. When the test over, calculate the

weight of each sample, also measure height, diameter, and water content of the sample. Time consolidation curve plotted using both the time fitting method i.e. square root and the logarithm for finding time taken for 90 % consolidation and 50 % consolidation respectively. After completion of the test void ratio versus loading (pressure) is plotted in a semi-log plot (i.e. e-log p plot) for both the bentonite and local soil at a different temperature.

Chapter 4

Result and Discussion

4.1 Swelling Pressure Test Result

Graph of swelling pressure with time from the initiation of water (distilled) supply at a different temperature plotted in Figure 4.1. From the graph, it was observe that initially there is a very rapid increase in swelling pressure and it reaches a nearly maximum value at 500 minutes after that it continues with very little fluctuation. Swelling pressure becomes constant approximately at five days. The maximum value of swelling pressure obtained for Bikaner bentonite at a different temperature shown in Table 4.1. Moisture content and degree of saturation before the test as well as after completion of the test also shown in the same table.

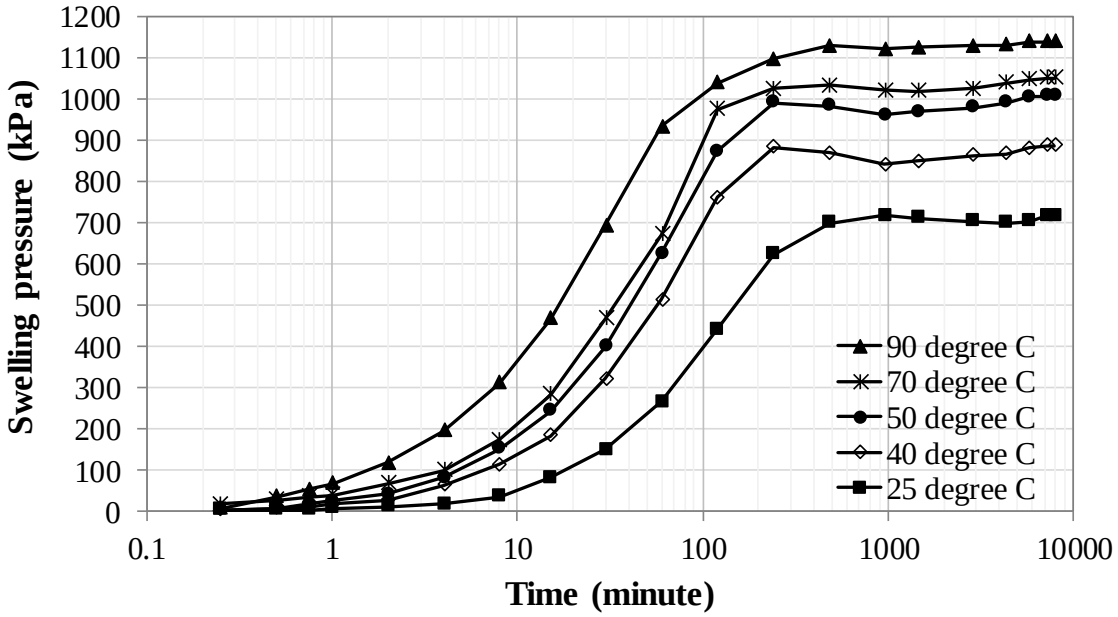


Figure 4.1 Time vs. swelling pressure plot of Bikaner bentonite at different temperatures.

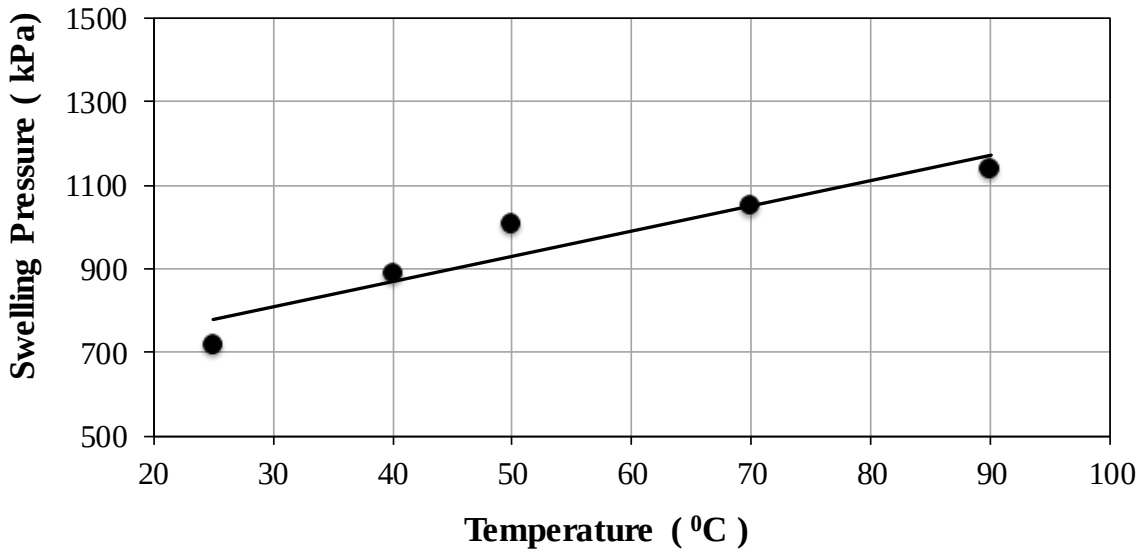


Figure 4.2 Swelling pressure variation with temperature for bentonite soil (dry density = 1.6 Mg/m^3)

As temperature increases swelling pressure of bentonite also increases (Pusch et al., 1990 in case of Na-bentonite, Cho et al., 2000 in case of Ca-bentonite). The reverse of this statement also reported by various researchers (Pusch et al., 1990 in case of Ca-bentonite, Lingnau et al., 1996 in case of Na-bentonite, and Villar and Lloret, 2004 for FEBEX bentonite in case of Ca and Mg bentonite). In this research work swelling pressure of bentonite measured at different temperatures from 25 ^{0}C to 90 ^{0}C and plotted in the graph shown in Figure 4.2. From the graph, it is clear that as temperature increases swelling pressure also increases. This increase in swelling pressure with temperature increase can explain using three processes i.e. change in hydration, osmotic, and pore water pressure. The hydration pressure, as we know it decreases very rapidly because of increase in temperature and hence on the surface of the bentonite water layer thickness reduces (Pusch et al., 1990). Because of temperature change, the degree of saturation effect the migration of moisture in the clay mineral and developed swelling pressure. Due to less water layer available on the mineral surface at a higher temperature, the double layer thickness reduces and hence the osmotic pressure increases (Pusch et al., 1990, Mohamed et al., 1992). Pore water pressure increases since there is a differential expansion of the pore water and the skeleton due to increasing in temperature in different parts of the clay mineral (Borgesson and Pusch, 1987).

Table 4.1 Swelling pressure test result on bentonite soil at different temperatures

Temperature (^{0}C)	Maximum Swelling Pressure (kPa)	Before Test		After Test	
		Moisture content (%)	Degree of saturation (%)	Moisture content (%)	Degree of saturation (%)
25	718	15	61	43.5	96
40	888	15	61	43.8	95
50	1008	15	61	43.7	97
70	1050	15	61	45.0	93
90	1139	15	61	46.2	97

Swelling pressure test result on bentonite soil at different temperatures shown in Table 4.1. Moisture content and degree of saturation before the test as well as after test also shown in the same table. From the table, it was found that swelling pressure becomes maximum when the degree of saturation about 95% for all temperature ranging between 25 and 90 ^{0}C.

4.2 Compressibility Test Result

Settlement of bentonite and local soil with respect to time (minute) is plotted in a graph using both the square root and logarithm of time fitting method. The graph plotted corresponding to the loading of 8.8 kg/cm^2 (863 kN/m^2) at different temperatures. Figure 4.3, 4.4, 4.5, and 4.6 shows that there is an increase in the settlement rate of soil as the test temperature increases. From these graphs, it was found that as temperature increases the rate of settlement of both the soil increases at the same applied load. The rate of settlement in the case of bentonite soil much higher than the local soil. The reason could be bentonite consists of the higher amount of clay soil, and the settlement rate mainly depends on the amount of clay soil present.

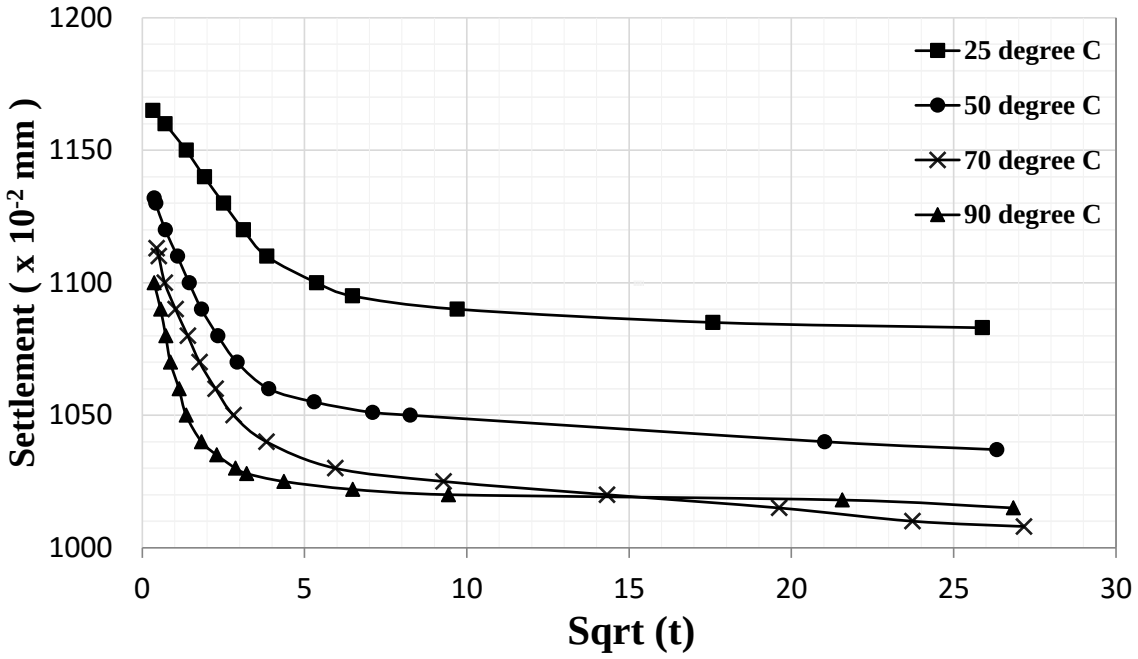


Figure 4.3 Settlement rate of bentonite clay using square root of time fitting method at different temperatures.

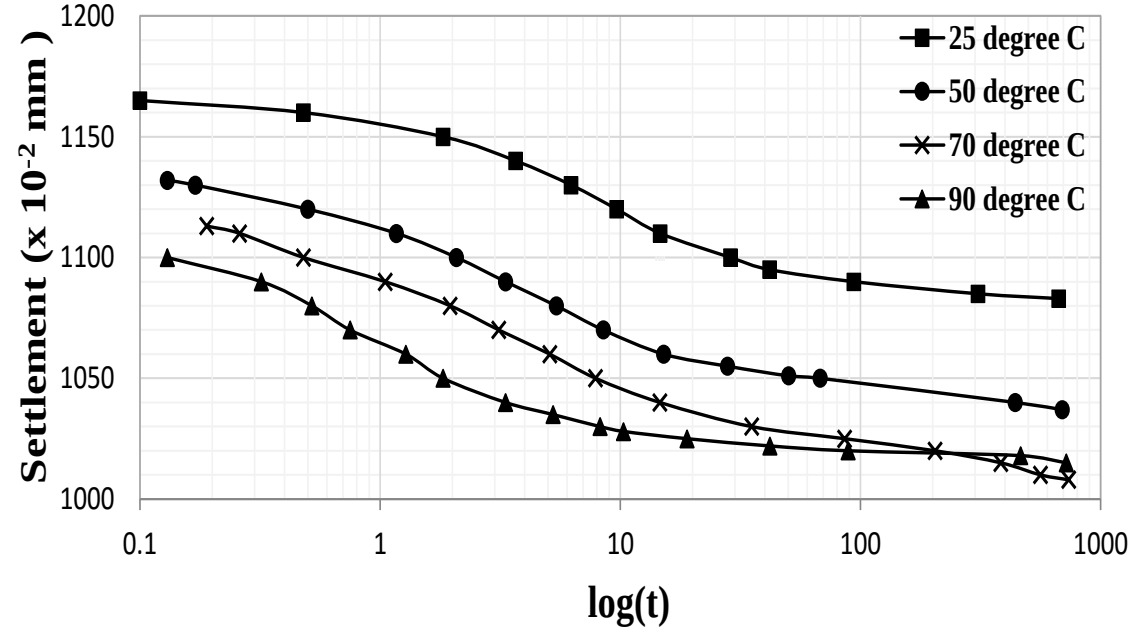


Figure 4.4 Settlement rate of bentonite clay using Logarithm of time fitting method at different temperatures.

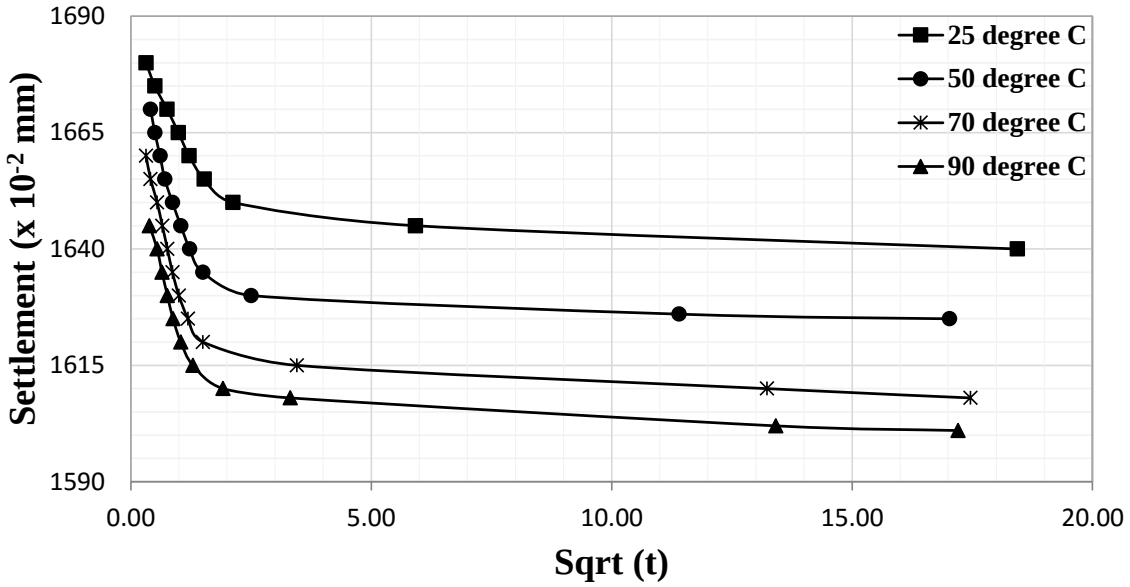


Figure 4.5 Settlement rate of local soil using the square root of time fitting method at different temperatures.

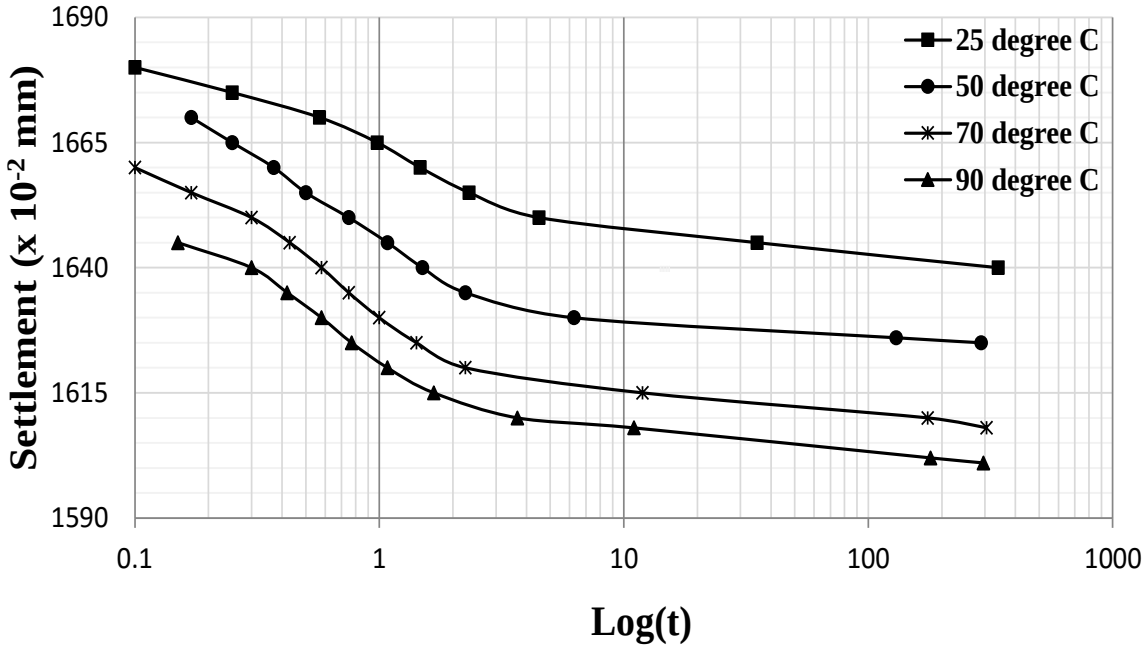


Figure 4.6 Settlement rate of local soil using Logarithm of time fitting method at different temperatures.

The effects of temperature on the mechanical properties observed are often contradictory. Cui et al. (2000) described two phenomena produced during heating: (1) expansion of soil constituents (solid and water): (2) mechanical weakening of the contacts

between soil aggregates. Fang and Daniels, 2006 measured the volume change and compressibility of clay soil at a different temperature. They reported that a) with an increase in temperature compressibility of soil increases, because of this the compression index Cc value, increased drastically and b) there will be an instant change in volume due to temperature increases, this magnitude of volume change mainly depends on the change in temperature. Saix et al. (2000) reported a decrease in compression index with the increase in temperature. On the other hand Romero et al. (2003) reported an increase in compression index with the increase in temperature for compacted Boom clay.

Void ratio corresponding to various loading is obtained and plotted in a semi-log plot of bentonite soil. The increase of slopes in loading (compression index C_c) with increasing temperature observed in e-log p curve of bentonite soil (Figure 4.7) means that compressibility of bentonite increases with increase in temperature. Main reason is that water viscosity decreases at elevated temperature, as viscosity decrease the permeability of clay increase. Habibagahi (1977) and Towhata et al. (1993) observed the higher value of permeability at elevated temperature because of reduction in the water viscosity. Hence, the consolidation test was slightly fast at high temperature.

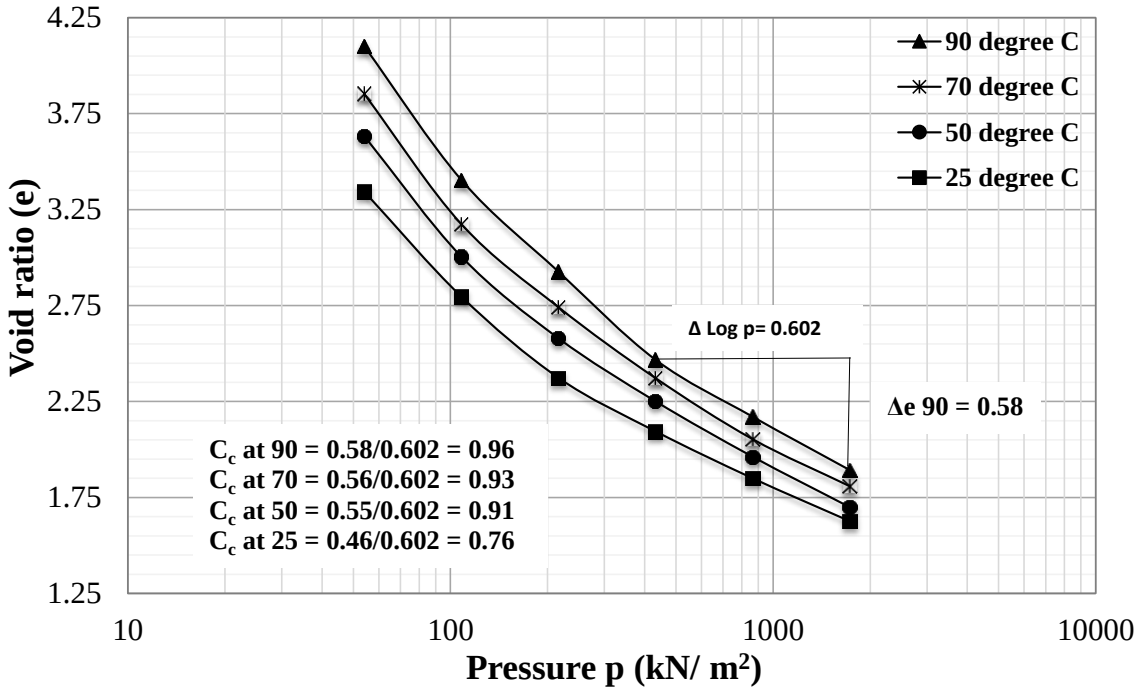


Figure 4.7 e-log p plot for Bikaner bentonite soil at different temperatures.

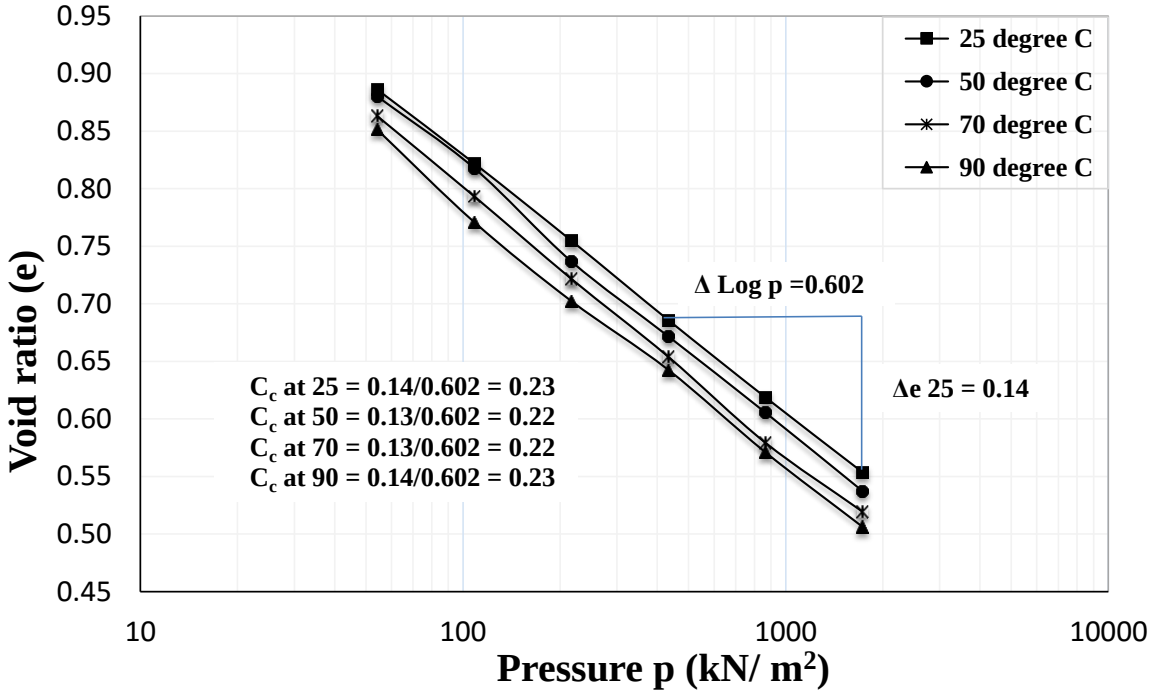


Figure 4.8 e-log p plot for Rourkela local soil at different temperatures.

In the case of Rourkela local soil e-log p plot (Figure 4.8) at different temperature were slightly moved in parallel trend. It shows that the compression index C_c (slopes of the normal consolidation lines) of the soil can be considered independent from temperature. Main reason is that Rourkela local soil content Kaolinite or illite and their compression index value is independent of temperature.

Figure 4.9 and 4.10 shows the coefficient of volume compressibility (m_v) versus consolidation pressure (p) for bentonite and local soil respectively. From the graph plotted for bentonite and local soil, it can conclude that as the consolidation pressure increases m_v value decrease for both the soil. It was observed that there is more or less no temperature effect on the m_v value of bentonite soil. From the graph plotted for local soil (Figure 4.10) it was also observed that at low pressure with the increase in temperature m_v value increase while at higher pressure there will be no temperature effect on m_v value.

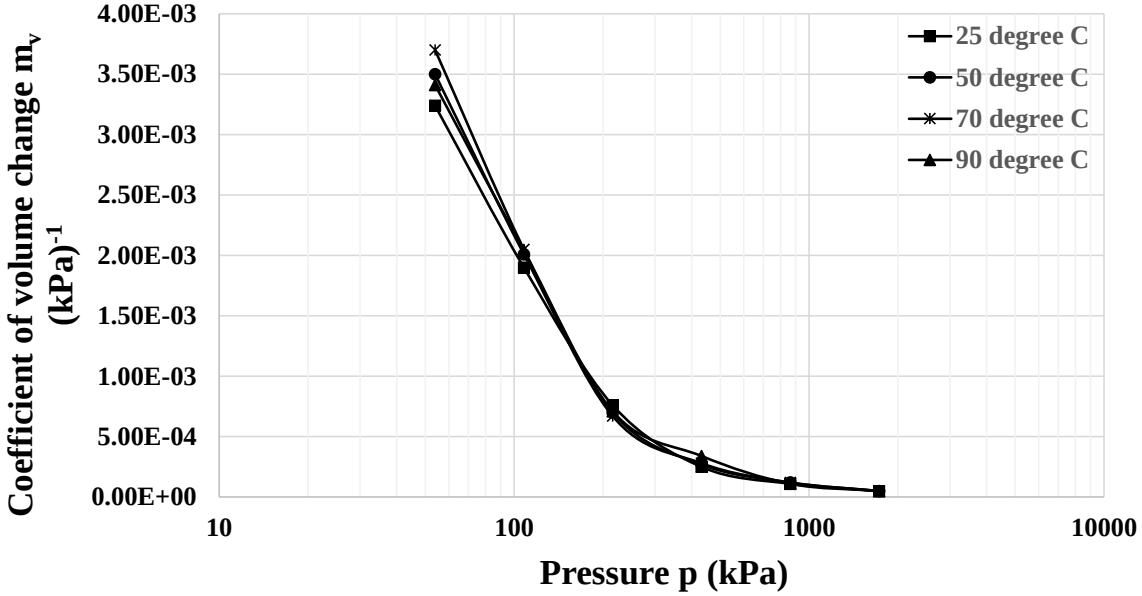


Figure 4.9 m_v -log p plot for Bikaner bentonite soil at different temperatures.

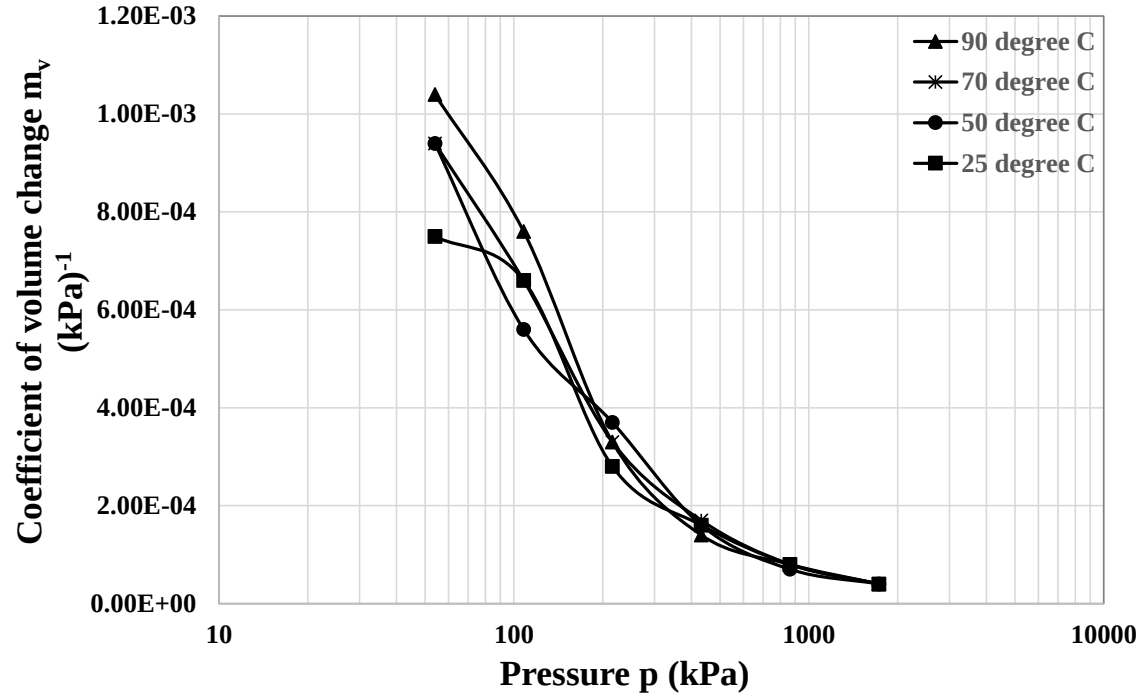


Figure 4.10 m_v -log p plot for local soil at different temperatures.

Table 4.2 Consolidation test on bentonite soil at different temperatures

Temperature (^{0}C)	Compression index C_C	Before Test			After Test		
		Moisture content (%)	Void ratio	Degree of saturation (%)	Moisture content (%)	Void ratio	Degree of saturation (%)
25	0.76	146	4.26	91	49	1.63	79
50	0.91	143	4.71	80	45	1.70	70
70	0.93	147	5.06	77	44	1.81	69
90	0.96	146	5.25	74	43	1.89	61

Table 4.3 Consolidation test on local soil at different temperatures

Temperature (^{0}C)	Compression index C_C	Before Test			After Test		
		Moisture content (%)	Void ratio	Degree of saturation (%)	Moisture content (%)	Void ratio	Degree of saturation (%)
25	0.23	33.2	0.97	91	16.0	0.55	59
50	0.22	34.7	0.98	94	15.4	0.54	76
70	0.22	34.4	0.96	96	15.2	0.52	78
90	0.23	32.8	0.96	91	14.0	0.51	73

The Moisture content, void ratio and degree of saturation value before the test as well as after test for Bikaner bentonite soil shown in Table 4.2 and for Rourkela local soil is shown in Table 4.3.

The variation of water content and degree of saturation with temperature for bentonite soil shown in Figure 4.11 and local soil is shown in Figure 4.12. From the graph obtained for bentonite soil (Figure 4.11) it was observed that temperature increase causes a decrease in the degree of saturation of the soil. From the graph obtained for local soil (Figure

4.12) it was observed that temperature increase causes first increase then decrease in the degree of saturation of the soil. It was also observed that there will be no effect of temperature on the final water content of both bentonite and local soil.

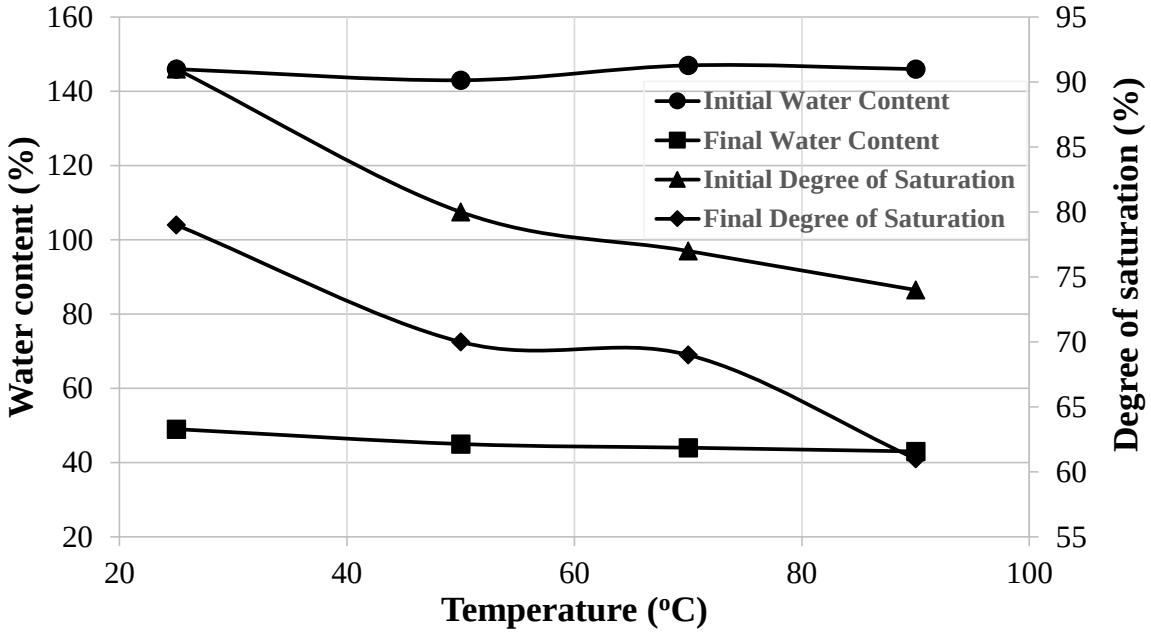


Figure 4.11 Water content and degree of saturation variation with temperature for bentonite soil.

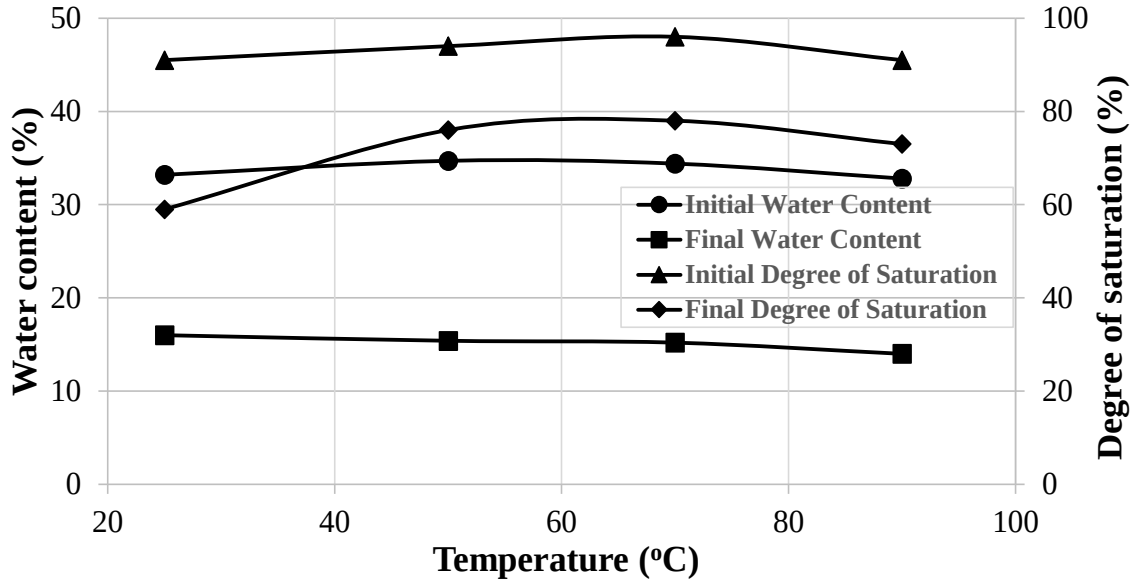


Figure 4.12 Water content and degree of saturation variation with temperature for local soil.

Chapter 5

Conclusions and Scope for Further Study

5.1 Conclusions

This work presented an experimental investigation to assess the swelling pressures and compressibility characteristics of soil as a function of temperature. A newly developed modified oedometer was used here to conduct swelling pressure and the compressibility of soil with temperature control. Compacted saturated bentonite at a targeted dry density of 1.6 Mg/m^3 were used to measure swelling pressure at different temperatures using constant volume method. Also conducted compressibility test for bentonite and local soil using same equipment with the saturated condition by using water content 20 % of the liquid limit of soil. Effects of water content and degree of saturation before the test as well as after test on the swelling pressure and consolidation also considered. Following findings may be drawn from the obtained results.

The temperature increase causes an increase in swelling pressure of bentonite soil. The measured value of swelling pressures at 25, 40, 50, 70, and 90 ^0C are 718, 888, 1008, 1050, and 1139 kPa respectively. The maximum value of swelling pressure occurs when the degree of saturation value near about 95 % i.e. full saturation condition.

The temperature increase causes an increase in compressibility index (C_c) of bentonite soil, however; there was no temperature effect on local soil. It also observed that increase in temperature increases C_c value for soil having higher clay content. In the case of bentonite soil, it was found that as consolidation pressure (p) increases the coefficient of volume change (m_v) value decreases. However, there is more or less no temperature effect on m_v value. It also found that with the increase in temperature degree of saturation decreases, however, there will be no temperature effect on the final water content of the bentonite soil.

In the case of local soil, it was found that as consolidation pressure (p) increases Coefficient of volume change (m_v) value decreases, also at low consolidation pressure as temperature increases m_v value increases however at higher consolidation pressure there is

no effect of temperature on m_v value. It was also found that as temperature increases the degree of saturation first increases then decreases. However, there will be no temperature effect on the final water content of the soil.

Few researchers earlier have shown opposite effect of temperature on swelling pressure and compressibility. The effect of temperature depends on the material type and the type of exchangeable cations presence in it. Many contributions have provided here for a better understanding of the behaviors of engineered clay barriers and effect of temperature on it.

5.2 Scope for Further Study

There is a vast scope of bentonite as a buffer and backfilling material. This study is limited to the only effect of temperature on swelling pressure and compressibility characteristics of soil. Several factors which effect bentonite soil could not be addressed here due to time limitation. So the further study should incorporate the following aspects in detail:

- Role of suction components on the swelling pressure of compacted bentonite.
- Swelling pressure test at different targeted dry density.
- Swelling pressure using diffuse double layer theory.
- Effect of voids on the swelling behavior of compacted bentonite-sand mixture.
- Effect of higher pressure on swelling and compressibility characteristics of bentonite.
- Stern layer effect on the compressibility behavior of bentonite.

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